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BEHAVIOUR OF SINGAPORE ROCKS UNDER UNIAXIAL AND TRIAxIAL LOADING

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Abstract

This report describes the strength and deformation behaviour of Jurong Formation sedimentary rocks in Singapore. With the need to develop future projects in this rock formation, the characteristics and design parameters for these rocks should be known. In this research, uniaxial and triaxial compressive tests as well as indirect tensile tests had been carried out on different sedimentary rocks, i.e. siltstone, limestone, sandstone and mudstone. During the tests, the axial and lateral deformations of the rocks were monitored against the stress level. In addition, acoustic emission and change in ultrasonic velocities during loading were recorded as well.

From the results of the tests performed, the identification of crack initiation and propagation thresholds using the conventional stress-strain curve approach is more reliable compared to the acoustic emission detection and ultrasonic technique. In addition, the effect of confining pressure was found to be minimal on the various stress threshold ratios. *Hoek-Brown* and *Mohr-Coulomb* failure criterions were established for the various rock types for a range of confining stresses similar to the expected working stresses in these rocks. It is recommended that future investigation to be extended to higher confining pressures as well as improvements to the testing equipment and techniques.

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Chapter 1: Introduction

1.1 General

Due to the scarcity and high cost of land surface area, Singapore has looked into expanding its infrastructure to the underground. A well known example of such project is the Underground Ammunition Facility (UAF) (Fig 1.1 & 1.2), developed by Defence Science & Technology Agency (DSTA) in April 1999 (Innovation, 2002). The UAF is built into a former granite quarry in Mandai (Bukit Timah Granite), which has six times the strength of normal concrete. In addition, by constructing the facility underground, the builders saved 300 hectares - a land area equivalent to 400 soccer fields, or half the size of Singapore's Pasir Ris New Town.

Fig 1.2 – Excavation works of Underground Ammunition Facility

Fig 1.1 – Completed Underground Ammunition Facility (September 2001)

Underground space could offer advantages such as the following:-

- more efficient use of land;
- better environmental quality;
- strategic storage of essential resources; and
- a higher degree of safety.

Such space can be utilized for sewage treatment plants, oil and gas storage, shelter and recreation centres, water collection and storage, power stations, underground pumped storage, military installations and industrial waste repositories. With all these added advantages, it is worthwhile to develop underground facilities in other rock formations of Singapore.

The two main formations considered suitable to host underground caverns are the Bukit Timah Granite and the Jurong Formation (sedimentary). Several other studies were done, including a study on the feasibility of cavern development in the Jurong Formation (1995) and the feasibility of developing the Underground Science City at Kent Ridge (1997). After the successful completion of the UAF, the feasibility of an underground petroleum storage facilities in rock cavern below Jurong Island (2001) was investigated (Civil Engineering Research, 2002).

1.2 Objectives of Present Study

The only rock formation that has been studied in details is the Bukit Timah Granite, mainly because it is believed to be the best candidate for hosting the first cavern (UAF) in Singapore. To date, there are no detailed previous studies to determine the strength and deformation behaviour of other rock types in Singapore. With the need to develop future projects in these rock formations, the characteristics and design parameters for these rocks should be known.

The objectives of this study are as follows:-

- To understand the deformation behaviour of sedimentary rocks of Singapore under uniaxial and triaxial loading conditions;
- To establish failure and cracking criterion for a range of confining stresses similar to the expected working stresses in these rocks; and
- To determine the design deformation parameters for the different rocks.

1.3 Scope of Present Study

Uniaxial and triaxial compression tests as well as indirect tension tests will be carried out on different rocks of Singapore. During the tests, the axial and lateral deformations of the rocks will be monitored against the stress level. Acoustic emission and change in ultrasonic velocities during loading will be recorded as well. Petrographic description of the rocks will be provided through X-Ray Fluorescence (XRF) analysis.

1.4 Organization

This report is organized into 11 chapters as follows: -

Chapter 1 introduces the objectives and scope of this present study.

Chapter 2 provides a literature review on the geological and geotechnical features of Singapore.

Chapter 3 gives an insight of the two non-destructive testing methods; acoustic emission (AE) and ultrasonic testing (P&S waves).

Chapter 4 covers the site investigations that had been conducted at Jurong Island, which is part of Jurong Formation.

Chapter 5 gives an outline of the methodology of this study. Detailed experimental procedure will be covered as well.

Chapter 6, 7, 8 and 9 presents the experimental results obtained in the laboratory testing and discusses the findings on siltstone, limestone, sandstone and mudstone, respectively.

Chapter 10 compares and discusses the experimental results of the 4 rock types.

Chapter 11 concludes all main research findings and provides recommendations for further study.

References and appendices are attached at the end of the report.

Chapter 2: Literature Review

This chapter will provide a background on the geological and geotechnical features of Singapore. Some of the past researchers' work and what had been achieved will be highlighted as well. This will give us a better understanding on the behaviour of Singapore rocks.

2.1 Background

Singapore is a city state on an island located at the southern tip of Peninsular Malaysia, with a total land area of 682.3 km², which comprises one main island (604.2 km²) and a number of islets scattered off its north-east and south. In year 2000, the total population of Singapore was approximately 4 million. The city consists of a Central Business District (CBD) at the central south, over ten residential new towns across the island, industrial estates located mainly at Jurong in the southwest, and a nature reserve at Bukit Timah in the central part of the island.

2.2 General Geology of Singapore

The rocks of Singapore can be broadly classified into four main geological formations (Fig 2.1), namely:-

- a) the igneous rocks consisting of the *Bukit Timah Granite* and the *Gombak Norite* in the north and central-north;
- b) the sedimentary rocks of the *Jurong Formation* in the west and southwest;
- c) the Quaternary deposits of the *Old Alluvium* in the east; and
- d) the recent deposits of the *Kallang Formation* of the alluvium member, the transitional member and marine clay that are distributed throughout the island.

Apart from the four main geological formations, there are also other smaller formations. They are as follows:-

- a) The *Sajahat Formation* which mainly consists of quartz sandstone in the north-eastern corner of the island. It was formed during the lower Paleozoic and is the oldest rock in Singapore.

- b) The *Tekong Formation* which is made up of beach sands and gravels of littoral and fluvial origin along the northeast and southwest coasts.
- c) The *reclaimed land* which is generally sandfill on top of the existing geological materials along the east, south and west coasts.

Fig 2.1 – Simplified geological map of Singapore (after PWD, 1976)

Since rock formations are of interest in this study, only the *Bukit Timah Granite* and *Jurong Formation* will be discussed in details. A brief summary will be given for the *Old Alluvium* and *Kallang Formation*.

2.3 Bukit Timah Granite & Gombak Norite

In recent years, extensive investigations had been conducted on the Bukit Timah Granite in attempt to design and construct underground structures in Singapore (Zhao et al., 1994; Zhao, 1996). The following section will briefly describe the engineering geology of Bukit Timah Granite and Gombak Norite (Sharma et al., 1999).

2.3.1 Geological Setting of Igneous Rocks

The Bukit Timah Granite, which is an acidic igneous rock formed during the Triassic period (about 200 million years ago) is the base formation of Singapore. About one-third of the Singapore Island and the whole of Pulau Ubin is the outcrop of this formation. It varies from granite through adamellite to granodiorite, and is usually light grey and medium to coarse-grained. The main minerals are quartz (30%), feldspar (60 ~ 65%), biotite and hornblende.

The Gombak Norite, which appears in the western edge of Bukit Timah Granite, can be seen at Little Guilin Lake in Bukit Batok Town Park (Fig 2.2a & 2.2b). Little Guilin was formerly a quarry (now flooded) and is so named because the rocky scenery there resembles that at Guilin in China. These noritic and gabbroic rocks are coarse-grained and rich in plagioclase with varying amount of clino- and orthopyroxene minerals in intergranular texture. These rocks also exhibit high compressive strength.

Fig 2.2a & 2.2b – Gombak Norite at Little Guilin Lake in Bukit Batok Town Park

2.3.2 Geo-Engineering Properties of Igneous Rocks

The geological and engineering properties of the granite and norite are as follows:-

- a) Weathering is very extensive, with the depth of weathering up to 70 m.
The soil profile is typically tropical and humid.
- b) Rock material has a very high compressive strength (average of 190 MPa).

- c) There are 4 to 5 joint sets, the dominant one being sub-vertical with NNW-SSE strike.
- d) Permeability of the rock mass is very low. Groundwater flow is only likely in fractured zones and faults.
- e) High horizontal stress in NNE-SSW direction due to tectonic movement.

2.3.3 Properties of Bukit Timah Granite

Typical properties of the weathered Bukit Timah Granite are shown in Table 2.1 (Sharma et al., 1999). From the table, it can be observed that the mechanical and engineering properties vary significantly at different weathering grades.

Table 2.1 – Properties of weathered Bukit Timah Granite (Sharma et al., 1999)

Weathering Grade	ρ_{bulk} (g/cm ³)	SPT N-value	RQD (%)	$I_{s(50)}$ (MPa)	σ_c (MPa)	v_p (km/s)	E_d (GPa)	k (x 10 ⁻⁸) (m/s)
Residual soil (VI)	2.09	13	-	-	-	1.2	4.5	1.0*
Completely weathered (V)	2.14	33	-	-	-	0.9	5.4	-
Highly weathered (IV)	2.32	> 100	45	1.8	32	0.7	5.4	5.12
Moderately weathered (III)	2.43	-	83	5.6	88	4.8	33.2	1.80
Slightly weathered	2.54	-	96	9.9	165	5.6	56.2	1.59
Fresh granite	2.66	-	99	11.1	192	5.8	60.3	0.58
Note: ρ_{bulk} = Bulk Density; SPT = Standard Penetration Test; RQD = Rock Quality Designation; $I_{s(50)}$ = Point Load Index; σ_c = Uniaxial Compressive Strength; v_p = Velocity of Compression Wave; E_d = Dynamic Modulus of Elasticity; k = Rock Mass Permeability. * In the normally consolidated range								

2.3.4 Rock Mass Quality of Bukit Timah Granite

Rock mass quality can be influenced by various parameters and boundary conditions. The two most commonly used rock mass classification systems are:-

- a) Q-system proposed by Norwegian Geotechnical Institute (NGI) (Bieniawski, 1989); and
- b) RMR (Rock Mass Rating) system (Bieniawski, 1989).

The Q-system is empirical and is based on evaluation of a large number of case histories of underground excavation stability, and is an index for the determination of the tunnelling quality of a rock mass. The numerical value of this index Q is defined by:

$$Q = \left(\frac{RQD}{J_n} \right) \times \left(\frac{J_r}{J_a} \right) \times \left(\frac{J_w}{SRF} \right) \quad (2.1)$$

where

- RQD = rock quality designation, for degree of jointing;
- J_n = number of joint sets;
- J_r = joint roughness;
- J_a = joint alteration or infilling;
- J_w = water pressure or leakage; and
- SRF = stress reduction factor.

The above six parameters can be estimated from site investigations and verified during excavation. One point to note is that the Q-value is a function of only three parameters, which are crude measures of:

- a) Block size (RQD / J_n);
- b) Inter-block shear strength (J_r / J_a); and
- c) Active stress (J_w / SRF).

Table 2.2 shows the range of Q-values and their respective rock mass quality description.

Table 2.2 – Q-System Rock Mass Classification (Bieniawski, 1989)

Q-value	Class	Rock Mass Quality Description
400 ~ 1000	A	Exceptionally Good
100 ~ 400	A	Extremely Good
40 ~ 100	A	Very Good
10 ~ 40	B	Good
4 ~ 10	C	Fair
1 ~ 4	D	Poor
0.1 ~ 1	E	Very Poor
0.01 ~ 0.1	F	Extremely Poor
0.001 ~ 0.01	G	Exceptionally Poor

The evaluation of Bukit Timah granite using the Q-system indicates Q-values in the range of 10 ~ 400, except for a small number of weak zones. These results imply that about 98% of logged length of the rock mass can be classified as *Good* (Q-value 10 ~ 40, Class B) and above, and 77% of the logged length can be classified as *Very Good* (Q-value 40 ~ 100, Class A) and *Extremely Good* (Q-value 100 ~ 400, Class A). Furthermore, rock mass quality assessed through seismic reflection survey by P-wave velocities gave a Q-value in the order of 200, which agrees with that of the Q system (Sharma et al., 1999).

2.4 Jurong Formation

After the successful construction of Singapore’s first underground cavern, the UAF in Bukit Timah Granite, investigators had since looked into the potential of cavern development in Jurong Formation. The following section will discuss the geology and properties of sedimentary rocks in Jurong Formation.

2.4.1 Geological Setting of Jurong Formation

The Jurong Formation covers the western and south-western part of the Singapore Island. It is mainly made up of sedimentary rocks from the upper Triassic and lower to middle Jurassic age (230 ~ 180 million years ago). Some of these rocks include mudstone, shale, sandstone, conglomerate and limestone.

The main mineral composition includes quartz grains, clay minerals, feldspars and other altered igneous rock minerals. Low grade metamorphism is observed.

Based on litho-stratigraphic units or facies, the Jurong Formation is divided into six facies members (PWD, 1976), namely:-

- a) Queenstown facies (red to purple mudstone and sandstone);
- b) Jong facies (roundstone conglomerate and sandstone);
- c) Ayer Chawan facies (dark coloured marine muddy sandstone and mudstone);
- d) Rimau facies (quartz conglomerate and quartz sandstone);
- e) St. John facies (marine muddy fine sandstone); and
- f) Tengah facies (muddy marine sandstone).

The above six facies are named in accordance to their sequence of deposition, i.e. Queenstown being the oldest and Tengah being the youngest.

Fontaine and Lee (1993) reported a limestone unit at Pandan Reservoir, which was later named Pandan Limestone. Subsequently, limestone was also discovered at Pasir Panjang, Tuas, Buona Vista, Jurong Point and Kent Ridge.

2.4.2 Weathering of Jurong Sedimentary Rocks

The Jurong Formation has been subjected to intensive chemical weathering due to the humid tropical climate of Singapore. Sandstones became weaker and more porous, and mudstones had been reduced to clay after being subjected to weathering. The mineral composition of rocks is the key factor affecting the rate of weathering.

Guo and Zhao (1998) had observed different weathering schemes in carbonate rocks and they had proposed a weathering classification system with six rock mass weathering stages:-

- unweathered stage (stage I);
- initial weathering stage (stage II);
- intermediate weathering stage (stage III);

- late weathering stage (stage IV);
- last weathering stage (stage V); and
- residual stage (stage VI).

Using the above classification system, the limestone at Pandan and Tuas can be classified as intermediate weathering stage (stage III), while the carbonate rocks at Pasir Panjang may fall in the range of intermediate weathering stage (stage III) to last weathering stage (stage V).

2.4.3 Mechanical Properties of Jurong Sedimentary Rocks

The rock strength of Jurong Formation has been largely dependent on the extent of weathering subjected to the rocks. This is illustrated by the point-load strength index shown in Table 2.3.

Table 2.3 – Index Properties of Jurong Sedimentary Rocks (Zhao et al., 1999)

Rock Type	Seismic velocity (m/s)		Weathering grade	Point-load strength index $I_{s(50)}$ (MPa)
	P wave	S wave		
Conglomerate	1200 ~ 3000	350 ~ 900	I ~ II III ~ IV	3 ~ 12 (strong to very strong) 0.1 ~ 3 (weak to moderately strong)
Sandstone	1250 ~ 3000	500 ~ 800	I ~ II III ~ IV	3 ~ 12 (strong to very strong) 0.1 ~ 3 (weak to moderately strong)
Siltstone	1250 ~ 2000	300 ~ 700	I ~ II III ~ IV	3 ~ 12 (strong to very strong) 0.1 ~ 3 (weak to moderately strong)
Slaty shale/ phyllite	1250 ~ 2000	300 ~ 700	I ~ II III ~ IV	0.3 ~ 6 (moderately weak to moderately strong) <0.1 ~ 0.3 (very weak to weak)
Limestone/ marble	2000 ~ 4500	1000 ~ 2500	I ~ II III ~ IV	3 ~ 12 (strong to very strong) 0.1 ~ 3 (weak to moderately strong)

In summary, slightly weathered to fresh (Stage I ~ II) conglomerate, sandstone, siltstone and carbonate rocks are classified as strong to very strong ($I_{s(50)} = 3 \sim 12$ MPa), while the highly to moderately weathered (Stage III ~ IV) varieties are classified as weak to moderately strong ($I_{s(50)} = 0.1 \sim 3$ MPa). The only exception is the slaty shale/ phyllite. In its slightly weathered to fresh stage, it is

considered as moderately weak to moderately strong ($I_{s(50)} = 0.3 \sim 6$ MPa), whereas in highly to moderately weathered stage, it is very weak to weak ($I_{s(50)} = <0.1 \sim 0.3$ MPa). One point to note is that rock strength does not increase with depth as weathering is observed up to a depth of 80 m. It is believed that weathering took place along well-developed fractures in fault zones.

Limited numbers of uniaxial compression test were conducted by NTU in the past to determine some indicative properties of Jurong sedimentary rocks. These tests suggested that the Jurong sedimentary rocks are generally strong. This implies that the stability of underground excavation in these rocks will generally be structural rather than stress controlled (Sharma et al., 1999).

2.4.4 Rock Mass Quality of Jurong Sedimentary Rocks

The rock mass quality of Jurong Formation has been assessed using the Q-system through extensive geological investigation and boreholes analysis. About 47% of the logged rock cores are rated as fair ($Q = 4 \sim 10$) to good ($Q = 10 \sim 40$) or very good ($Q = 40 \sim 100$) quality rocks. Limestone (e.g. Pandan), conglomerate and sandstone (e.g. NTU campus, Labrador Park) are considered good quality rocks. Fair quality rocks are distributed throughout the Jurong area. Poor quality ($Q < 4$) rocks are mostly mudrocks or highly faulted rocks.

2.5 Old Alluvium

The Pleistocene (2 ~ 7 million years ago) deposits of the Old Alluvium covers the eastern part of the Singapore Island. The Quaternary deposits are underlain by the extension of the Bukit Timah Granite. The Old Alluvium consists of semi-consolidated sand and fine gravel with silt and clay lenses. The thickness of these Quaternary deposits varies from a few ten meters to more than 100 meters.

2.6 Kallang Formation

The Holocene (8000 years ago) sediments of the Kallang Formation is distributed around the Singapore Island and is underlain by the rock formations. These deposits cover much of the coastal plain, offshore zone and the deeply incised river valleys which penetrate to the centre of Singapore Island.

2.7 Compressive Strength of Rock

The compressive strength of rock, which is usually referred to as the Uniaxial Compressive Strength (UCS, σ_c), can be easily determined by using a cylindrical specimen loaded axially between platens of a compression testing machine. The UCS of that particular specimen is given by the relationship:

$$\sigma_c = \frac{F}{A_0} \quad (2.2)$$

where σ_c = Uniaxial compressive strength (MPa);
 F = Maximum load carried by the specimen (kN); and
 A_0 = Original cross-sectional area (mm²).

Although the determination of UCS is simple, it can be affected by *internal factors* such as mineralogy, grain size and porosity. These are the intrinsic properties of rocks which determine its strength. In general, higher quartz (binding material) content, finer grain size and lower porosity will result in higher compressive strength.

In addition, the following *external factors* will also affect the test results significantly:-

- a) Friction between platens and end surfaces;
- b) Specimen geometry:-
 - i) Shape
 - ii) Aspect ratio (L/d ratio)
 - iii) Size
- c) Rate of loading;
- d) Environment.

The above mentioned factors must be recognized and clearly defined before the test; else the results may be misleading and virtually useless. These factors are dealt with in the following section.

2.7.1 Friction between Platens and End Surfaces

Due to the “Poisson’s effect”, a specimen tends to expand laterally (both in radial and circumferential direction) as it shortens. However, frictional constraint at the planes of contact between the platens and end surfaces tends to prevent expansion. As a result, the specimen is not in a state of uniaxial compression in this region.

At the contact surfaces, the “effect of friction introduces a triaxial state of stress” (Vutukuri et al., 1974). Some of the measures to reduce this “End effect” include using platens of the same diameter as the specimen and introducing capping materials in between the surfaces.

2.7.2 Specimen Geometry

The geometry of a specimen can affect its compressive strength considerably due to “end effect” and stress distribution within.

2.7.2.1 Shape

Test specimens can be cylindrical, prismatic or cubic, but cylindrical shape is preferred due to its shorter preparation time and symmetrical stress distribution about the axis. Moreover, rock samples are usually cored using diamond cutter during borehole investigation.

2.7.2.2 Aspect Ratio (L/d ratio)

Stress distribution in specimens with small L/d ratio (short specimen) tends to be triaxial which reflect very high compressive strength. On the other hand, specimens with large L/d ratio (long specimen) fail due to elastic instability. As such, International Society for Rock Mechanics (ISRM) Committee on Laboratory Tests (1979a) suggested an L/d ratio of 2.5 to 3.0.

Specimens with this medium range L/d ratio are elastically stable and experience a more uniform stress distribution.

2.7.2.3 Size

It was observed that the strength of specimens usually decreases with the increase in size. This can be due to the increasing probability of flaws in the specimens. Theoretically, the size of the specimen should depend on the size of grains. ISRM Committee on Laboratory Tests (1979a) suggested cylinders having a diameter preferably of not less than NX core size, approximately 54 mm. The diameter of the specimen should be related to the size of the largest grain in the rock by the ratio of at least 10:1.

2.7.3 Rate of Loading

The compressive strength of rocks will usually increase with the rate of loading. This observation can be explained due to the time allowed for crack propagation. At lower loading rate, the compression test will take a longer time to reach failure, and this period of time actually allows the cracks within the specimen to propagate. This causes a lower compressive strength and a more ductile and mild failure mode. On the contrary, a higher loading rate will not allow the cracks to propagate as much as the former, thus a higher strength and a more brittle and violent failure mode are observed.

For static purposes, the rate of loading should be within certain limits so that the time required for conducting tests is reasonable. ISRM Committee on Laboratory Tests (1979a) suggested that failure shall occur within 5 to 10 minutes of loading; alternatively the stress rate shall be within 0.5 to 1.0 MPa/s.

2.7.4 Environment

Environmental factors, such as moisture, liquids and temperature, do have a significant effect on the compressive strength of rocks.

2.7.4.1 Moisture

Moisture has a varying effect on the compressive strength of rocks. It is noted that rocks exhibit a higher compressive strength in its dry state than in its saturated state. In view of this, the values obtained in laboratory tests must be corrected to in situ conditions for design purposes, else catastrophic failures can occur. However, it is time consuming to condition the specimens to bring it back to its original state, thus saturated specimens are recommended as it provide more conservative results.

ISRM Committee on Laboratory Tests (1979a) suggested that the prepared specimens shall be stored prior to testing for 5 to 6 days in an environment of 20 ± 2 °C and 50% humidity. However, the samples used in this study are tested in room temperature condition to maintain consistency. The moisture condition shall be reported in accordance with ISRM Committee on Laboratory Tests (1979b).

2.7.4.2 Liquids

The compressive strength of rocks is influenced by liquids contained in them. Liquids coming into contact with the rocks will decompose certain minerals, thus creating more liquid-filled voids. This process is more prominent at crack tips, whereby the increasing stress at the apex tends to accelerate the propagation of the cracks. Liquids which wet the surfaces of the rock invariably decrease the surface energy of the rock and hence its strength.

Therefore, it is recommended that in the preparation of specimens, cutting oils should be avoided and the use plain water should suffice.

2.7.4.3 Temperature

Compressive strength of rocks will increase with drying temperature due to its lower moisture content (as discussed in section 2.7.4.1). Normally, tests are conducted at room temperatures. If the in situ conditions are different, the tests should be conducted in that simulated environment.

2.8 Tensile Strength of Rock

Specimens tested for compressive strength often fail due to the development of tensile stresses. Since rocks are much weaker in tension than in compression, it is therefore important to look into the tensile strength of rocks in detail as well. Brittle failure theory predicts a ratio of compressive strength/tensile strength of about 8:1 but in practice, it is generally between 15:1 and 25:1.

The main difficulty in the direct tensile test is the gripping of specimens to achieve a uniform tensile stress distribution, without introducing bending stresses. Specially prepared specimens are required to overcome this problem, however, they are difficult and time consuming to prepare. As a result, a number of indirect methods have been developed to determine the tensile strength of rocks. An example is the Brazilian Test which will be carried out in this study.

2.8.1 Brazilian Test

In the Brazilian test, a rock cylinder of length (L) and diameter (D) is loaded with a load (P) in a diametrical plane along its axis. The sample usually fails by splitting along the line of diametrical loading and the tensile strength (σ_t) can be obtained from

$$\sigma_t = \frac{0.636 P}{DH} \tag{2.3}$$

- where
- σ_t = Tensile strength (MPa);
 - P = Load at failure (N);
 - D = Diameter of test specimen (mm); and
 - H = Thickness of test specimen measured at centre (mm).

Mellor and Hawkes (1971) stated that the Brazilian test is useful for brittle materials but for other materials, the test may give totally invalid results.

There are various factors which will affect the Brazilian tests' results. They are as follows:-

- a) Specimen geometry;
- b) Rate of loading;
- c) Environment; and
- d) Mineralogy and porosity.

There are certain specifications to adhere to in order to obtain reliable Brazilian tests' results. Past studies done on these aspects will be reviewed in the following section.

2.8.1.1 Specimen Geometry

In general, for a given diameter specimen (D), the tensile strength of the rock will decrease with the increase in thickness (H). Apart from selecting a convenient geometry for the experiment, it is also desirable to standardize dimensions to facilitate comparison of data. It should be noted that the effect of size is greater in tensile than compression because in tension, cracks open and give rise to large strength reduction, whereas in compression, the cracks close and so disturbances are appreciably reduced.

Mellor and Hawkes (1971) suggested that the minimum specimen diameter to be NX core size (54 mm) and the minimum thickness to be the specimen radius. ISRM Committee on Laboratory Tests (1978a) also adopts these specifications for the Brazilian test.

2.8.1.2 Rate of Loading

Mellor and Hawkes (1971) did Brazilian tests on NX cores of two rock types over a range of loading rates from 16.67 to 30,000 N/s. In general, higher tensile strength is observed with higher loading rate. It is a similar

situation for both compressive and tensile failure due to the crack propagation phenomenon (section 2.7.3).

ISRM Committee on Laboratory Tests (1978a) suggested that failure in the weakest rocks shall occur within 15 to 30 seconds and a loading rate of 200 N/s is recommended.

2.8.1.3 Environment

Dube and Singh (1972) studied the effect of moisture content on tensile strength of five different types of sandstones. Their results show a decrease in the strength ranging from 11 to 48% in fully saturated state compared to its dry state.

ISRM Committee on Laboratory Tests (1978a) suggested that the water content shall be measured and reported in accordance with the method suggested by ISRM Committee on Laboratory Tests (1979b).

2.8.1.4 Mineralogy and Porosity

These are the intrinsic properties controlling the rock strength. Merriam et al. (1970) found that the tensile strength of granite rocks is inversely proportion to the quartz content. They attributed this relationship to textural differences, whereby high-quartz rocks composed of equidimensional grains exhibiting little crystal interlocking. On the other hand, low-quartz rocks consist of interlocking laths and prisms.

Dube and Singh (1972) observed an increase in tensile strength with percentage of matrix (cementing material) in sandstones. In addition, they also reported a decrease in strength with an increase in porosity.

2.9 Strength of Rock under Triaxial Stresses

Natural rock formations are usually subjected to triaxial stresses and the strength of rock under such state is important in calculating the bearing capacity of foundation rocks and in the design of underground rock caverns. In addition, failure and cracking criteria for a range of confining stresses similar to the expected working stresses in these rocks can be established with the results from triaxial tests.

Von Karman (1911) is the pioneer investigator for the properties of rocks under a triaxial stress. Since then, triaxial test is the most common method, whereby a jacketed cylinder specimen is subjected to an axial force and lateral fluid-confining pressure in a triaxial cell (Fig 2.3). The stress state in this test is limited to a special case in which two of the principal stresses (σ_3) are equal.

Fig 2.3 – Cutaway view of triaxial cell (after Hoek and Franklin, 1968)

2.9.1 Triaxial Compressive Strength

Triaxial tests have to be carried out if the complete failure envelope of a rock material is required and it is done in accordance with the method suggested by ISRM Committee on Laboratory Tests (1978b).

There are various factors affecting the triaxial compressive strength of the rocks such as the following:-

- a) Confining pressure;
- b) Aspect ratio (L/d ratio);
- c) Loading path;
- d) Temperature;
- e) Pore pressure; and
- f) Rate of loading.

2.9.1.1 Confining Pressure

Von Karman (1911) found that as the confining pressure increases, the compressive strength increases as well. Subsequently, many researchers also reported similar findings. However, there is a limit to the increase in strength. As the confining pressure increases further, the corresponding increase in strength becomes less and less. At high confining pressures, the strength is nearly constant (Schwartz, 1964).

Ryabinin et al. (1971) pointed out that brittle rupture takes place only up to a definite confining pressure known as the “threshold pressure”. Beyond this threshold, the rock material converts from a brittle state into a plastic one. The plasticity of the rock will increase with pressure.

The relationship between confining pressure (σ_3) and axial stress (σ_1) at failure is linear for many rocks (Bodonyi, 1970) and can be represented by the equation

$$\sigma_1 = \sigma_c + m \sigma_3 \quad (2.4)$$

where σ_c = uniaxial compressive strength for specimen having L/d ratio of 2;

m = constant.

However, the actual laboratory results show a curved σ_1 - σ_3 relation. Thus, empirical criterions are proposed, and one of the most widely used is the *Hoek-Brown failure criterion*, which is developed for application in underground excavation design. The experimental results are fitted using the following criterion (Hoek et al., 1995):

$$\frac{\sigma_1}{\sigma_c} = \frac{\sigma_3}{\sigma_c} + \left(m \frac{\sigma_3}{\sigma_c} + s \right)^a \quad (2.5)$$

where σ_1 = axial principal stress;

σ_3 = confining principal stress;

σ_c = uniaxial compressive strength of intact rock material;

m , s and a are parameters that depend on the characteristics of the rock mass. For reasonable quality rock mass, the value of the parameter a is equal to 0.5, and the parameter s is equal to 1.0 for intact rock. Table 2.4 shows the recommended values for m and s for rock types of different rock mass quality (Hoek et al., 1995).

Table 2.4 – Hoek-Brown Rock Mass Strength Criterion – *m* & *s* value (Hoek et al., 1995)

$$\frac{\sigma_1}{\sigma_c} = \frac{\sigma_3}{\sigma_c} + \left(m \frac{\sigma_3}{\sigma_c} + s \right)^a$$

Hoek-Brown Rock Mass Strength Criterion	Carbonate rocks with well developed crystal cleavage. (Dolomite, limestone and marble)	Lithified argillaceous rocks (Mudstone, siltstone, shale and slate) (normal to cleavage)	Arenaceous rocks with strong crystals and poorly developed crystal cleavage (Sandstone and quartzite)	Fine grained polyminerallc igneous crystalline rocks (Andesite, dolerite, diabase and rhyolite)	Coarse grained polyminerallc igneous and metamorphic crystalline rocks (Amphibolite, gabbro, gneiss, granite, Norite and quartz-diorite)
Intact rocks	m = 7.0 s = 1.0	m = 10.0 s = 1.0	m = 15.0 s = 1.0	m = 17.0 s = 1.0	m = 25.0 s = 1.0
Very good quality rock mass	m = 3.5 s = 0.1	m = 5.0 s = 0.1	m = 7.5 s = 0.1	m = 8.5 s = 0.1	m = 12.5 s = 0.1
Good quality rock mass	m = 0.7 s = 0.004	m = 1.0 s = 0.004	m = 1.5 s = 0.004	m = 1.7 s = 0.004	m = 2.5 s = 0.004
Fair quality rock mass	m = 0.14 s = 0.0001	m = 0.20 s = 0.0001	m = 0.30 s = 0.0001	m = 0.34 s = 0.0001	m = 0.50 s = 0.0001
Poor quality rock mass	m = 0.04 s = 0.00001	m = 0.05 s = 0.00001	m = 0.08 s = 0.00001	m = 0.09 s = 0.00001	m = 0.13 s = 0.00001
Very poor quality rock mass	m = 0.007 s = 0	m = 0.01 s = 0	m = 0.015 s = 0	m = 0.017 s = 0	m = 0.025 s = 0

2.9.1.2 Aspect ratio (L/d ratio)

In unconfined compression tests, end effects due to small L/d ratio can affect rock strength considerably. However, Mogi (1966) showed that end effects noticeably disappear with the increasing confining pressure in triaxial tests. He found that the failure strength at 0.1 MPa confining pressure is greatly affected by L/d ratio, but at higher pressure, it becomes independent of the ratio.

In order to standardize the testing procedure, ISRM Committee on Laboratory Tests (1978b) suggested an L/d ratio of 2.5 to 3.0.

2.9.1.3 Loading Path

There are basically two loading procedures that are commonly used for triaxial testing: proportional loading and constant confining stress loading. In proportional loading, the confining stress and the axial stress are applied in a fixed ratio. In constant confining stress loading, the confining stress is applied completely, before any significant axial stress had been applied, and then kept constant as the specimen is loaded axially.

Swanson and Brown (1971) reported that the maximum stresses in all rocks tested are independent of the stress loading path. ISRM Committee on Laboratory Tests (1978b) suggested the proportional loading path method whereby the confining pressure and axial load are increased simultaneously until the predetermined confining pressure is reached. Subsequently, the confining pressure is maintained and the axial load is applied at a constant stress rate such that the failure will occur within 5 to 15 min of loading.

2.9.1.4 Temperature

The effect of temperature on the triaxial strength of rocks is important to access its fire resistance in the case of a fire breakout in underground rock caverns. Griggs et al. (1960) investigated a number of rocks at various temperatures. From Fig 2.4, it can be seen that the effect of temperature on different rocks varies. The effect is more pronounced on calcite than on silicate igneous rocks. Dolomite remains almost unaffected by rise in temperature. At 800 °C, dolomite is considerably stronger than basalt though at lower temperatures (below 500 °C), basalt is stronger than dolomite.

Fig 2.4 – Strength of rocks and minerals at 500 MPa confining pressure.
Note logarithmic scale of ordinate (Griggs et al., 1960)

In summary, the triaxial compressive strength of rocks decreases with the rise in temperature, although the effect varies with different types of rocks.

2.9.1.5 Pore Pressure

In routine triaxial testing of rock, it is not usual to measure pore pressures. The pore pressure will counter the effect of confining pressure. In other words, if the pore pressure is equal to the confining pressure, then fracture occurs at a constant value of the deviation stress, which is equal to the unconfined compressive strength. This illustrates the “effective confining pressure” concept whereby

$$\sigma_3' = \sigma_3 - u \quad (2.6)$$

where σ_3' = Effective confining pressure (MPa);
 σ_3 = Applied confining pressure (MPa); and
 u = Pore water pressure (MPa).

However, this concept is applicable only to porous rocks as reported by Schwartz (1964). He studied the effect of pore pressure on the triaxial strength of limestone, sandstone (void ratio = 0.20), granite and marble (void ratio = 0.02). The first two rocks are, therefore, very porous while the other two are almost impervious. Results showed that strength of porous rocks is essentially the same when tested dry and saturated, when compared using the concept of “effective confining pressure” (Fig 2.5). On the other hand, the strength of impervious rocks shows no correlation with the “effective confining pressure” (Fig 2.6).

Fig 2.5 – Pore pressure tests for Pottsville sandstone (Schwartz, 1964)

Fig 2.6 – Pore pressure tests for Stone Mountain granite (Schwartz, 1964)

In summary, the effect of pore pressure depends on the porosity of the rock, viscosity of pore fluid, specimen size and rate of loading. Generally, the increase of pore pressure will decrease the strength of porous rocks. The effective confining pressure is valid when loading rates is less than some critical value which depends on the permeability of the rock, viscosity of pore fluid and specimen size.

2.9.1.6 Rate of Loading

The triaxial compressive strength of rocks usually increases with an increase in rate of loading. ISRM Committee on Laboratory Tests (1978b) suggested that the axial load is applied at a constant stress rate of 0.5 to 1.0 MPa/s such that the failure will occur within 5 to 15 min of loading.

2.9.2 Shear Strength of Rock from Triaxial Tests

A series of triaxial tests at various confining pressures (σ_3) are to be done on specimens from the same rock and the corresponding axial stress (σ_1) at failure is recorded. With all these data, a family of Mohr's circles and common tangents are drawn giving the Mohr's envelope. The shear strength of the rock, τ can be determined at any point on the enveloping curve with the corresponding stress, σ . The slope (ϕ) of the enveloping curve gives the angle of internal friction of the rock (Fig 2.7).

Fig 2.7 – Mohr's envelope for rock

Alternatively, ISRM Committee on Laboratory Tests (1977) suggested plotting the confining pressures as abscissa and the axial stresses as ordinates (Fig 2.8). A straight line is fitted through these data points and this line is characterized by calculating its gradient, m and its vertical intercept, b . With these two parameters, the angle of internal friction (ϕ) and a value for the “apparent” cohesion C (Coulomb’s failure theory) can be calculated using the following formulae:

$$\phi = \sin^{-1} \left(\frac{m-1}{m+1} \right) \quad (2.7)$$

$$C = b \frac{1 - \sin \phi}{2 \cos \phi} \quad (2.8)$$

Fig 2.8 – Strength envelope (ISRM, 1978)

2.10 Brittle Fracture Mechanism

Brittle failure is regarded as the sudden loss of cohesion across a plane that is not preceded by an appreciable permanent deformation. It may occur in rock on both microscopic and macroscopic scales. Several researchers had extensively studied the deformation and fracture characteristics of brittle rocks and their past studies have shown that the failure process can be sub-divided into various stages on the stress-strain characteristics through axial and lateral deformation measurements recorded during uniaxial and triaxial laboratory tests. Brace (1964) and Bieniawski (1967) later defined these various stages as follows: -

- crack closure;
- linear elastic deformation;
- crack initiation and stable crack growth;
- critical energy release and unstable crack growth; and
- failure and post-peak behaviour.

Fig 2.9 – Stress-strain diagram showing the definition of crack initiation, crack damage and peak strength (Eberhardt et al., 1998)

2.10.1 Crack Closure, σ_{cc}

Crack closure occurs during the initial stage of loading where the applied stress (σ) is greater than the crack closure stress (σ_{cc}) as shown in Fig 2.9. During crack closure, the stress-strain response is non-linear and the extent of this non-linear region is dependent on the initial crack density and geometrical characteristics of the crack population. Linear elastic deformation takes place when the majority of pre-existing cracks have closed.

2.10.2 Crack Initiation, σ_{ci}

This is the stress level where micro-cracking initiates. Brace et al. (1966) and Bieniawski (1967a) defined crack initiation occurs at the point where the lateral and volumetric strain curves depart from linearity as shown in point 'A' in Fig 2.10. Martin (1993) later defined crack initiation as the stress level at which dilation (crack volumetric strain increase) begins after experiencing a plateau of crack volume unchanged during elastic deformation as shown in Fig 2.9.

Fig 2.10 – Relationship between axial, lateral and volumetric strain (Brace et al., 1966; Bieniawski, 1967a; Latjtai & Latjtai, 1974)

Crack volume can be determined by subtracting the linear elastic component of the volumetric strain, $\varepsilon_{\text{elastic}}$ given by: -

$$\varepsilon_{\text{elastic}} = \frac{1 - 2\nu}{E} \sigma_{\text{axial}} \quad (2.9)$$

where E and ν are the elastic constants Young's modulus and Poisson's ratio respectively, and σ_{axial} is the axial stress level, from the volumetric strain calculated from the measured axial (ε_a) and lateral (ε_l) strains, given by: -

$$\varepsilon_v = \varepsilon_a + 2\varepsilon_l \quad (2.10)$$

The remaining volumetric strain (ε_{vcr}) is attributed to axial cracking, i.e.

$$\varepsilon_{\text{vcr}} = \varepsilon_v - \varepsilon_{\text{elastic}} \quad (2.11)$$

2.10.3 Crack Coalescence, σ_{cs}

Once crack initiation has taken place, stable crack propagation follows. The actual mechanism of stable fracture propagation is dependent on the type of crack arrays. Bieniawski discovered that crack propagation will cease and become stabilized when the crack path is parallel to the major principal stress direction.

Eberhardt et al. (1998) concluded that rate changes (in terms of lateral strain) occur during stable crack propagation because crack growth is predominantly in the direction of uniaxial loading. When the load is continually increased, additional cracks begin to grow and increase in size. The cracks eventually begin to interact with one another and coalesce at oblique angles to the loading direction. At this stage, it will result in an increased rate of change in the volumetric stiffness curve.

2.10.4 Crack Damage, σ_{cd}

During this stage, the lateral strain rate change exceeds the rate change in axial strain, thereby causing a change of the volumetric strain curve in a reverse direction. Bieniawski (1967b) suggested that the unstable crack growth occurs

at the point of reversal in the volumetric strain curve and is also known as the point of critical energy release or crack damage stress threshold (σ_{cd}) as shown as point 'B' in Fig 2.10. At this stage, crack growth velocity will take control of the propagation process and will continue to cause rupture of the specimen even if the applied load is held constant.

Since a certain degree of subjectivity is required in picking up the point of volumetric strain reversal directly from the volumetric strain curve, Eberhardt et al. (1998) proposed using the volumetric stiffness, $V_{stiffness}$, which is given by: -

$$V_{stiffness} = \frac{\sigma_{axial}}{\epsilon_V} \quad (2.12)$$

where σ_{axial} is the axial stress level and ϵ_V is the corresponding volumetric strain.

From the plot of volumetric stiffness versus axial stress as shown in Fig 2.11, the point of crack damage stress threshold (σ_{cd}) is very distinct. Further examination of the volumetric stiffness curve can indicate a few other stress thresholds.

Fig 2.11 – Break down and correlation of volumetric stiffness with stages in the compressive failure process of Lac du Bonnet granite (Eberhardt et al., 1998)

2.10.5 Peak Strength, σ_f

The peak strength of the rock is achieved when numerous micro cracks have coalesced and the rock can no longer support an increase in load. However, Martin (1993) discovered that peak strength of granite is not a unique material property but is dependent on the loading conditions such as the loading rate. On the other hand, the crack initiation and crack damage stresses were found to be more characteristic and are essentially independent of the loading conditions.

Chapter 3: Acoustic Emission & Ultrasonic Studies

In this study, apart from investigating the conventional mechanical properties of the sedimentary rocks, acoustic emission (AE) and the change in ultrasonic velocities during loading will be investigated as well. The following chapter will give an insight of these two non-destructive testing methods.

3.1 General

Acoustic emission and ultrasonic studies provide a non-destructive analysis of microcrack formation, orientations and mechanisms and their effect on the mechanics of a test sample. This concept can be applied to any material that deforms through brittle fracturing and is common in the testing of rock, concrete and metals. Microcracks in laboratory samples are formed as a function of the material properties. Furthermore, the coalescence of microcracks into macrofractures will cause major damage to a sample and eventually lead to failure. Through AE and ultrasonic studies, damage to the sample can be investigated and failure mechanisms understood.

3.2 Acoustic Emission

Acoustic emission is a naturally occurring phenomenon, which has been observed as in earthquakes, and rock burst in mines in the early days (Drouillard, 1996). According to ASTM, an acoustic emission refers to “the generation of transient elastic waves during the rapid release of energy from localized sources within a material”. This may be caused by the dislocation, grain boundary movement, or initiation and propagation of fractures through and between mineral grains in polycrystalline rock.

The technology of AE began way back in the 1950s with the work of Joseph Kaiser. During the 1950s and ‘60s, many researchers looked into the fundamentals, developed specific instrumentation, and characterized the AE behaviour of many materials. One unique capabilities of AE is that it is a non-destructive test (NDT) method for monitoring dynamic processes. In the 1970s, working groups, such as

the Acoustic Emission Working Group (AEWG), were formed to coordinate research activities. Computers became a basic component for instrumentation and data analysis in the 1980s. Today, waveform-based acoustic emission analysis has been proven to be a useful tool, especially in identifying the significant cracking process of brittle rock. However, traditional acoustic emission system is still facing the difficulty in separating valid signals from extraneous noise.

In the following section, fundamentals of AE wave-formed analysis are reviewed. Subsequently, the literature on the application of AE in the study of failure mechanism is discussed. New approach of AE data analysis – modal analysis is introduced to eliminate extraneous noise and its theory and principles are illustrated.

3.2.1 Fundamentals of Acoustic Emission Testing

Some of the fundamentals of AE testing, including the phenomenon of AE, characteristics of AE techniques, AE testing system and characteristics of AE signals, will be covered in the following section.

3.2.1.1 Acoustic Emission Phenomenon

Obert and Duvall (1942) discovered that a stressed rock pillar appeared to emit micro-level sounds when they were carrying out sonic studies in a deep hard rock mine. It was later verified that this phenomenon, which is often referred to as “rock talk”, is a measure of the mechanical stability of a rock material or structure. Through the “rock talk” or AE, as it is now more commonly known, engineers are now able to understand better what the stressed rock is “saying” (Obert and Duvall, 1942).

There are two acoustics techniques, which are commonly used in geotechnical area (Hardy, 1981). The first one, shown in Fig 3.1a, is the *ultrasonic technique*, which utilizes two transducers; a transmitter (T) to generate a mechanical signal within the material under study, the other a receiver (R) to monitor the changes of the transmitted signal. This method also allows the measurement of the *Ultrasonic Pulse Velocity* (UPV) within

the material and this will be discussed in further details at the later part of this chapter. On the contrary, the *AE technique* only requires a receiving transducer to monitor self-generated signals occurring within the material (Fig 3.1b). Most acoustic emission signals contain both a compression (p-wave) and shear (s-wave) component.

Fig 3.1 – Two acoustic techniques utilized to evaluate behaviour of geologic structures; (a) Ultrasonic technique and (b) Acoustic emission technique (Hardy, 1981)

3.2.1.2 Acoustic Emission Non-Destructive Testing

AE testing has been known for its non-destructive testing capabilities to monitor structural integrity, detect leaks and failures in mechanical equipment, and characterise materials behaviour (Spanner, 1987). There are two significant differences between AE and other non-destructive methods. Firstly, the energy detected is generated within the material (Fig 3.1b) rather than from an external source (transmitter), as in ultrasonic or radiography (Fig 3.1a). Secondly, the AE method is able to detect *dynamic* processes associated with the degradation of structural integrity. Some of these dynamic processes include crack growth and plastic deformation, which are the major sources of AE.

AE is a wave phenomenon whereby the attributes of particular waves are used to characterise the material. The two most regularly monitored

waveform parameters in AE tests are the frequency and amplitude. **Table 3.1** gives an overview of the factors that will influence AE response amplitudes (Spanner, 1987).

Table 3.1 – Factors that affect the relative amplitude of acoustic emission response (Spanner, 1987)

Factors that tend to increase acoustic emission response amplitude	Factors that tend to decrease acoustic emission response amplitude
High strength	Low strength
High strain rate	Low strain rate
Low temperature	High temperature
Anisotropy	Isotropy
Non-homogeneity	Homogeneity
Thick sections	Thin sections
Brittle failure (cleavage)	Ductile failure (shear)
Material containing discontinuities	Material without discontinuities
Martensitic phase transformations	Diffusion-controlled phase transformations
Crack propagation	Plastic deformation
Cast material	Wrought material
Large grain size	Small grain size
Mechanically induced twinning	Thermally induced twinning

3.2.1.3 Characteristics of Acoustic Emission Techniques

The AE test is a passive method that monitors the dynamic redistribution of stresses within a material or component. Therefore, acoustic emission monitoring is only effective while the material or structure is subjected to an applied stress. In this study, the AE will be monitored during the loading of the rock sample under uniaxial and triaxial conditions.

There are two important features of AE test, namely the *Kaiser effect* and the *felicity effect* that will be elaborated in the following sections.

a) Kaiser Effect

As mentioned earlier, Joseph Kaiser is one of the pioneer researchers in AE technology way back in the 1950s (Drouillard, 1996). He found that once the load has been applied and the occurrence of acoustic emission at that stress has ceased, additional acoustic emission will not occur until that stress level is exceeded, even if the load is completely removed and then reapplied. This irreversible response is, thereafter, named the *Kaiser effect*.

b) Felicity Effect

In fibre reinforced plastic components, emission is often observed at load lower than the previous maximum, especially when the material is in poor condition or close to failure (Spanner, 1987). As such, the term *felicity effect* is used to describe the breakdown of the Kaiser effect. In addition, the Kaiser effect fails most noticeably in situations where time dependent mechanisms control the deformation.

The *felicity ratio* is defined as the ratio between the applied load at which acoustic emission reappears during the next application of loading and the previous maximum applied load. Thus, the Kaiser effect may be considered as a special case of the felicity effect (a felicity ratio of 1).

3.2.1.4 Acoustic Emission Testing Systems

Some of the common components in an AE testing system include sensors, preamplifiers, filters and amplifiers to make the signal measurable. Fig 3.2 shows a block diagram of a generic four-channel AE system.

Fig 3.2 – Schematic diagram of a basic four-channel acoustic emission testing system (Spanner, 1987)

a) Acoustic Emission Sensors

Very minute movement of the surface molecules will occur when an AE wavefront impinges on the surface of the test object. The functions of a sensor are to detect this mechanical movement and convert it into a specific and usable electric signal. This sensor is usually a piezoelectric transducer which functions as an electro-mechanical conversion device.

The main considerations for sensor selection are (1) operating frequency; (2) sensitivity; and (3) environmental and physical characteristics. High frequency signals ($f > 2$ kHz) are usually detected using accelerometers, whereas that of low frequency ($f < 1$ Hz) are detected with displacement gauges. Signals in between these extremes ($1 \text{ Hz} < f < 2 \text{ kHz}$) can be easily picked up by velocity gauges. Typical geophones and accelerometers have sensitivities in the range of $40 \sim 400 \text{ V/m/s}$ and $2 \sim 100 \text{ mV/g}$, respectively.

b) Preamplifier and Frequency Selection

It is important for the preamplifier to be close to the sensor and this can be achieved by incorporating it into the sensor housing. The functions of a preamplifier are to provide the required filtering, gain (usually 40 dB) and

cable drive capability. The monitoring frequency for the AE test is defined by the filtering in the preamplifier and the type of sensors used.

The observed frequency spectrum of acoustic emission signals is significantly influenced by the resonance and transmission characteristics of both the specimen (geometry as well as acoustic properties) and the sensor. In practice, the lower frequency limit is governed by background noise. It is unusual to go below 10 kHz except in microseismic work. The upper frequency limit is governed by wave attenuation that restricts the useful detection range and it is unusual to go above 1 MHz. The most common frequency for acoustic emission testing in rocks is 100 ~ 300 kHz (Hardy, 1981).

c) System Mainframe

The first elements in the mainframe are the main amplifier and the thresholds, which are adjusted to determine the test sensitivity. Main amplifier gains in the range of 20 to 60dB are most commonly used. In a small portable instrument, acoustic emission events or threshold crossings may simply be counted and the count then converted to an analogue voltage for plotting on a chart recorder.

d) Signal Processing and Displays

In typical implementations, each acoustic emission signal is measured by hardware circuits and the measured parameters are passed through the central microcomputer to a disk file of signal descriptions. During or after data recording, the system extracts data for hardcopy reports and graphic displays such as plots of acoustic emission versus time or load.

3.2.1.5 Acoustic Emission Signal Characterisation

The purpose of signal characterisation is to identify the sources and to evaluate their significance using the sensor output waveform. Thus, there is a qualitative (source identification) and quantitative (source intensity or severity) aspect to characterisation.

A discrete or burst type AE can be represented by an idealised damped sinusoid shown in Fig 3.3. Although the signals are seldom simple waveforms, they usually rise rapidly to maximum amplitude and decay nearly exponentially to the level of background noise. A threshold detection level is set above the background level in order to detect an AE event (Fig 3.3).

Fig 3.3 – Definition of simple waveform parameters (Spanner, 1987)

Some of the signal characteristics which had been summarised by Spanner (1987) will be presented in the following.

a) Acoustic Emission Events and Count

The emission count is the number of times a signal crosses a preset threshold. High amplitude events of long duration tend to have many threshold crossings. The number of threshold crossings per unit time depends on (1) the sensor frequency, (2) the damping characteristics of the sensor, (3) the damping characteristic of the structure, and (4) the threshold level (Spanner, 1987).

The idealized damped sinusoid signal can be represented by Eq. 3.1:

$$V = V_o \exp(-Bt) \sin \omega t \quad (3.1)$$

where V = output voltage of sensor;
 V_o = initial signal amplitude;
 B = decay constant (greater than 0);
 t = time; and
 ω = angular frequency.

The *ring-down counts* is the number of times the sensor signal exceeds a counter threshold. When the time t required for the signal to decay to the threshold voltage V_t is long compared to the period of oscillation, then the number of counts N from a given event can be given by:

$$N = \frac{\omega}{2\pi B} \ln \frac{V_o}{V_t} \quad (3.2)$$

where V_t is the threshold voltage of the counter.

b) Correlation of Count Rates and Material Properties

As shown in Fig 3.3, a larger event requires more cycles to ring down to the trigger level and will produce more counts than a smaller event. This provides a measure of the intensity of the AE event. The correlations have been established between total counts, count rate and various fracture mechanics parameters such as stress intensity in Eq. 3.3 or fatigue crack propagation rate in Eq. 3.4.

$$N \sim K^n \quad (3.3)$$

where N = total number of counts;
 K = stress intensity factor; and
 n = constant with a value between 2 and 10.

$$\frac{dN}{dc} \sim \frac{da}{dc} \quad (3.4)$$

where N = total number of counts;
 a = crack size; and
 c = number of cycles.

The ring down counts are the complex function of the frequency response of the sensor and the structure (ω), the damping characteristics of the sensor, the event and the propagation medium (B), signal amplitude, coupling efficiency, sensor sensitivity, amplifier gain and the threshold voltage (V_o). Maintaining stability of these parameters from test to test is difficult but is essential for consistency of interpretation.

c) Acoustic Emission Event Energy

Since acoustic emission activity is attributed to the rapid release of energy in a material, the energy content of the acoustic emission signal can be related to this energy release. The true energy is directly proportional to the area under the acoustic emission waveform. The electrical energy U present in a transient event can be defined as:

$$U = \frac{1}{R} \int_0^{\infty} V^2(t) dt \quad (3.5)$$

where R is the electrical resistance of the measuring circuit. Direct energy analysis can be performed by digitising and integrating the waveform signal or by special devices performing the integration electronically.

d) Acoustic Emission Signal Amplitude

The peak signal amplitude can be related to the intensity of the source in the material producing an acoustic emission. The measured amplitude of the acoustic emission waveform is affected by the same test parameters as the event counts. For practical purposes, a simple equation can be used to relate signal amplitudes, events and counts:

$$N = \frac{Pf\tau}{b} \quad (3.6)$$

where N = cumulative ring down counts;
 P = cumulative event counts;
 f = resonant frequency of the sensor;
 τ = decay time of the event; and
 b = amplitude distribution slope parameter.

3.2.1.6 Effects of Background Noise

Acoustic emission monitoring must always recognize the presence of noise. The most effective remedy for noise interference is to identify the noise sources and then remove or inhibit them. The sources of noise could be from the *hydraulic noise* of a leaking hydrostatic pressure vessel, *mechanical noise* from the friction movement of mechanical parts in contact with structure under test (Spanner, 1987). Noise should be eliminated either by mechanically removing the identified noise source from the test setup or by selectively analyzing the acoustic emission test data.

a) Hydraulic Noise

For modern servo hydraulic testing machine, serious noise problem could emerge during acoustic emission tests. Hydraulic noise will be transmitted to the acoustic emission sensor if the valves controlling the hydraulic loading are integral with the yoke of the machine. The source of the noise in the machine is the cavitations of hydraulic fluid in narrow channels of the load piston and cylinder (Spanner, 1987). This can be controlled by separating the servo controller from the test frame with high pressure hoses

b) Mechanical Noise

Noise signals generated by friction have relatively low frequency with low rise time compared to acoustic emission bursts from cracks that generally have a rise time of less than 25 microseconds if the sensor is near the source. *Master-slave technique* is commonly used to reject noise (Spanner, 1987).

Master sensors are mounted near the area of interest and are surrounded by slave or guard sensors relatively remote from the master. If the guard sensors detect a signal before the master sensors, then the event occurred outside the area of interest and is rejected by the instrument circuitry.

c) Filtering of Background Noise

Most acoustic emission monitoring is done using a frequency passband above 100 kHz. Fortunately, most acoustic noise diminishes in amplitude at these higher frequencies. Less noise is detected as the frequency passband becomes narrow and higher, but these frequencies can also inhibit acoustic emission detection. An unacceptable remedy for noise interference is to reduce the instrument gain or raise the detection threshold because this will also exclude valid acoustic emission signals.

As a rule, an indication of the level of interference from electrical or mechanical noise can be obtained by noting the noise level of the acoustic emission system at the test site with sensors on the test structure and comparing this ambient level in a low noise environment.

3.2.2 Acoustic Emission in Geotechnical Application

The AE technique can be applied to various kinds of geotechnical materials such as soils (sandy and clayey), rocks, fossilized deposits (coal) and ice. AE monitoring is useful in nearly every geotechnical application where subsurface deformation can be anticipated (Hopwood, 1987). Some of the field applications of AE include underground storage, slope stability, tunnels, hard rock mines, and coal mines (Hardy, 1981).

Diverse acoustic emission source mechanisms operate in geotechnical testing and the resulting stress waves are often transmitted through heterogeneous mediums. This led to a variety of testing techniques and systems based on the detecting frequency range shown in Fig 3.4.

Fig 3.4 – Frequency range of various types of acoustic emission studies. Circled numbers represent regions where studies were undertaken (Hardy, 1981)

3.2.2.1 Application of Acoustic Emissions in Rock Testing

Lockner (1993) had reviewed the application of acoustic emission in laboratory studies in the past. These applications could be divided into the following four categories.

- a) Simple counting of the number of acoustic emission events prior to sample failure shows a correlation between acoustic emission rate and inelastic strain rate. Additional sorting by amplitude has shown that acoustic emission events obey the power law of frequency-magnitude relation observed for earthquakes. These cumulative event count techniques are being used in conjunction with damage mechanics models to determine how damage accumulates during loading and to predict failure.
- b) Second area of research involves the location of hypocenters of acoustic emission source events. This technique requires precise arrival time data of acoustic emission signals recorded over an array of sensors that are essentially a miniature seismic net. Analysis of the spatial and temporal variation of event hypocenters has improved our understanding of progression of microcrack growth and clustering leading to rock failure.

c) Third area of study involves the analysis of full waveform data as recorded at receiver sites. It can determine fault plane solutions of acoustic emission source events from first motion data.

d) P and S wave velocities (including spatial variations) and attenuation have been obtained by artificially generating acoustic pulses which are modified during passage through the sample.

3.2.2.2 Acoustic Emission Signatures of Various Rock Types

Boyce et al. (1981) had compared Mogi (1962) AE signature curve to Bieniawski's (1967a) concept of the brittle fracture process in rock. Fig 3.5 illustrates that the closing of cracks can be associated with region A-B, the linear elastic deformations with region B-C, the stable fracture propagation with region C-D, and the unstable fracture propagation with region D-F.

Fig 3.5 – Generalized stress versus acoustic emission signature and possible relationship to mechanisms to brittle rock fracture (Boyce et al., 1981)

Boyce et al. (1981) had carried out unconfined compression tests on ten different types of rocks (Table 3.2).

Table 3.2 – Summary of behaviour of rocks tested (Boyce et al., 1981)

Designation	Rock Type	Unconfined Strength (MPa)	AE behaviour type
A	Dolomite	241	I
B	Shale	57	III
C	Mica schist	19	III
D	Gneiss	71	I
E	Marble	102	II
F	Dolomitic marble	96	I
G	Granite	160	I
H	Shale	168	II
I	Mica schist	67	III
J	Limestone	64	IV

They concluded that the AE signatures of these rock specimens was that Mogi’s pattern of stress versus AE behaviour is a general type, but not universal. They had proposed four types of behaviour as follows (Fig 3.6):

- a) *Mogi Type I* – characterised by crack closure, elastic, stable fracture, and unstable fracture zones;
- b) *Unstable Type II* – characterized by crack closure, elastic and unstable fracture zones.
- c) *Dense Type III* – characterized by elastic, stable fracture, and unstable fracture zones.
- d) *Dense, Unstable Type IV* – characterized by elastic and unstable fracture zones.

Fig 3.6 – Stress versus acoustic emission signatures for various types of rock (Boyce et al., 1981)

Montoto et al. (1981) investigated the AE signature of five uniaxial loaded granitic rocks and they revealed that there were three significant stages of AE behaviour, which is similar to *Dense – Type III* in Boyce's model (**Fig 3.6**). The absence of the AE in the initial stage is due to the rocks' low porosity and high compactness.

Fonseka et al. (1985) did some acoustic emission studies of crack development in three different rock types, i.e., micro-granodiorite, dolerite and marble. The AE characteristics can be subdivided into five regimes, which are different from those proposed by Boyce et al. (1981). However, the observed trends of cumulative ring down counts from both authors are similar.

Jones and Murrell (1989) investigated the AE characteristic of sandstones under high confining pressure. The AE count rate increases at an accelerating rate with strain near peak stress where cracking activity is dominated by microcrack initiation and extension. However, after reaching the peak stress, there is a transition from largely elastic behaviour to cataclastic flow. Acoustic emission rate drops at the peak stress plateau and then increases during cataclastic flow of dilatancy (Fig 3.7). There was no shear fault in specimen and specimen failed by cataclastic flow.

Fig 3.7 – Stress and acoustic compression wave speed as a function of strain in a specimen of dry Darley Dale sandstone at a confining pressure of 175MPa and room temperature (Jones and Murrell, 1989)

3.2.2.3 Stress Threshold Identification

Eberhardt et al. (1998) illustrated that the correlation between the behaviour of the volumetric stiffness curve and crack initiation, and damage stress thresholds could be validated through AE analysis.

Fig 3.8 – Plot of ring down counts versus axial stress for a 130 Level pink granite (Eberhardt et al., 1998)

As shown in Fig 3.8, the beginning of significant acoustic emission activity is at 80 MPa, which coincides with the crack initiation stress threshold (σ_{ci}) of 80 MPa as determined using the volumetric stiffness curve in Fig 3.9. The crack initiation could be determined by identifying the stress level at the start of the relative irregularities region in the volumetric stiffness curve.

Fig 3.9 – Plot of average volumetric stiffness vs. axial stress (Eberhardt et al., 1998)

The departure of linearity of volumetric stiffness curve prior to crack damage stress threshold (σ_{cd}) is explained as the coalescence of propagating cracks. This is evident in the acoustic emission data which show increasing bursts of acoustic emission activity leading up to (σ_{cd}) as shown in Fig 3.9.

3.2.3 Modal Analysis of Acoustic Emission Signals

Traditional acoustic emission instrument is facing difficulty in an attempt to separate signals due to crack growth from extraneous noise signals by utilizing event-based analysis of the acoustic emission signal such as rise time, pulse width and energy, etc (Hardy, 1981). Sondergeld et al. (1981) expressed the uncertainty of event duration determination especially in multiplicity of events and long duration sliding events.

Dunegan (1997) studied the waves present in plates of different thickness created by breaking pencil leads in-plane (IP) and out-of-plane (OOP) on the plates and by detecting the resulting stress waves with a high fidelity acoustic emission transducer mounted on the edge and surface of the plates. In his research, transducer and instrument were built and successfully detected both modes of the acoustic emission signals, which separated valid signals from extraneous noise and its interference.

3.2.3.1 Theory and Principle of Modal Analysis

From the research of Dunegan (1997), transducer and instrument in concert are designed to recognise *So* (zero order symmetrical Lamb wave), a high frequency shear wave, and *ao* (zero order asymmetrical Lamb wave) as shown in *Fig 3.10*.

Fig 3.10 – (a) Symmetrical Lamb wave (b) Anti-symmetrical Lamb wave (Dieulesaint and Royer, 1974)

Fig 3.10(a) shows that on both sides of the middle plane, the longitudinal components are equal and the shear components have opposite signs. Meanwhile, in Fig 3.10(b), on both sides of the meridian plane, the longitudinal component changes sign whereas the shear component does not.

The two primary sources of extraneous noise encountered when performing an acoustic emission test on large structures are due to impact and friction between members. Both of these noise sources are out-of-plane (OOP) sources which most of the energy is carried away from the source by **ao**, a low frequency dispersive flexure wave (less than 100 kHz).

Crack growth is an in-plane (IP) source and most of the energy is carried away from the source by **So**, a high frequency (greater than 100 kHz) extensional wave, and **Sh**, a high frequency (greater than 100 kHz) shear wave. If the depth in the plate where crack extension occurs has a moment arm with respect to the centre line of the plate, **ao** waves will also be produced with a magnitude dependent on the length of the moment arm (crack depth).

An example of these two different modes of signals simulated by breaking pencil lead at the edge and on the top of the plate of 500×375 mm aluminium plate with thickness of 5 mm are detected and illustrated in Fig 3.11 by Dunegan (1997).

Fig 3.11 – Detected high frequency and low frequency signals from OOP and IP Sources (Dunegan, 1997)

3.2.3.2 Extracting Crack Growth Signals from Noise

The innovation of Dunegan is that transducer is designed to operate as a displacement sensitive device in the 20 ~ 60 kHz range of frequencies which responds to the low frequency flexure wave produced by OOP sources in the 20 ~ 60 kHz range of frequencies, and as a velocity sensitive device at frequencies above 100 kHz which responds to the high frequency extensional waves produced by IP sources, and to the high frequency shear waves produced by both OOP and IP sources.

During the signal conditioning process, detected signals from transducers are split into two frequency bands: (1) 20 ~ 60 kHz bandpass (LF); (2) 100 kHz hi-pass (HF). Signals of each of these two frequency bands are analogue peak detected and read by an analogue-to-digital A/D converter. From the experiment performed by Dunegan (1997), extraneous noise (OOP sources) could easily be distinguished with amplitudes of HF/LF ratio of less than 1 and the HF/LF ratios for crack like signals (IP sources) are greater than 1 as shown in Fig 3.11.

3.3 Ultrasonic Studies

Laboratory ultrasonic velocity measurements have been used extensively to study the elastic behaviour of geological materials at simulated stress conditions. Ultrasonic testing is non-destructive and provides compression (P) wave and shear (S) wave velocity information which can be used in calculating dynamic elastic constants such as Poisson's Ratio (ν), Young's Modulus (E), Bulk Modulus (K), and Shear Modulus (G). The elastic constants are calculated based on the assumption that an intact, homogenous, isotropic specimen is being tested. In the past, laboratory determinations of shear and compression wave velocities were conducted using a complicated arrangement of oscilloscopes, signal pulse generators, and power supplies.

Data readings were typically taken from the oscilloscope display immediately at the test condition of interest and handwritten so that the next reading could be taken. Unfortunately, as the waveform could not be recalled for further analysis, the results were based entirely on the judgement of the operator at the test condition. The results could be questionable due to the difficulty in determining the "arrival time" if the waveform is not ideal.

3.3.1 Fundamentals of Measurement

The "first arrival" time of the wave must be determined in order to calculate the P and S wave velocities. The first arrival time can be defined as the amount of time that is taken for the waveform to propagate through the test specimen. As the piezoelectric crystals are mounted within the test platens, the time required for the P and S waves to pass through the platen faces must first be determined. This is done by performing a test with the platens in "face to face" contact with each other. These face to face delay times are then subtracted from the observed P and S first arrival times of the test specimens to give the correct first arrival times. The P or S wave velocity is then calculated as the length of the specimen divided by the first arrival time.

3.3.2 Application of Ultrasonic Tests in Rock Testing

The failure of brittle rocks under compression is mainly due to the three distinct stages of microcracking, namely formation, growth and coalescence. The presence of microcracks significantly decreases the ultrasonic wave velocities in the rock. This behaviour may therefore be used to monitor the progressive damage of the rock under increasing compressive stress.

Sayers (1990) investigated the variation of ultrasonic wave velocities in Berea sandstone under triaxial compression (Fig 3.12).

Fig 3.12 – P&S wave velocities in dry Berea sandstone. Note: Ox_3 is the major principal axis (Sayers, 1990)

P wave velocity, v_{33} increases monotonically with stress, while S wave velocities, v_{31} and v_{23} increase throughout most of the test except for a small decrease immediately prior to the peak stress. v_{11} , v_{22} (P wave) and v_{12} (S wave) increase initially, but then decrease after the first third of the test.

There is a common trend of increasing wave velocity in the initial stage of the test. This is due to the closure of pre-existing microcracks and no additional cracks are expected to occur at these low stress levels. As the stress increases, new microcracks are formed and pre-existing ones grow and coalesce, thus explaining the decrease of wave velocities during the later part of the test.

3.3.3 Stress Threshold Identification

Similar to the application of AE, the variation of ultrasonic wave velocities during compression test can be used to identify the various cracking stress threshold.

Fig 3.13 – Stress threshold identification using P&S wave velocities. Note: Ox₃ is the major principal axis

A typical ultrasonic wave velocity against stress graph can generally be divided into four main zones as shown in **Fig 3.13**. In *Zone I*, the wave velocity increases till the stress threshold of crack closure (σ_{cc}) due to closure of pre-existing microcracks as discussed earlier. The velocity will then maintain constant during elastic deformation (*Zone II*) till the stress threshold of crack initiation (σ_{ci}). Subsequently, the velocity will decrease due to the formation of new microcracks and the growth of pre-existing ones. In *Zone III*, the stress threshold of crack coalescence (σ_{cs}) could not be determined as there is no distinct feature exhibited. Finally, there is a change in the rate of decrease of the wave velocity at the stress threshold of crack damage (σ_{cd}) due to unstable crack propagation (*Zone IV*).

3.3.4 Calculation of Dynamic Elastic Constants

From the theory of elasticity for homogeneous and isotropic solids, the elastic constants can be determined with the sample density and the P and S wave velocities using the following expressions (GCTS, 1994):

$$E = \frac{\rho V_s^2 (3 V_p^2 - 4 V_s^2)}{V_p^2 - V_s^2} \tag{3.7}$$

$$G = V_s^2 \tag{3.8}$$

$$\nu = \frac{V_p^2 - 2 V_s^2}{2 (V_p^2 - V_s^2)} \tag{3.9}$$

- where E = modulus of elasticity (Pa);
- G = modulus of rigidity (Pa);
- ν = Poisson’s ratio;
- ρ = density (kg/m³);
- V_p = compression wave velocity (m/s); and
- V_s = shear wave velocity (m/s).

3.3.5 Dynamic Properties of Rocks and Minerals

Some of the known dynamic properties of rocks and minerals are shown in Table 3.3 (GCTS, 1994). This provides a reference when carrying out ultrasonic tests on the sedimentary rocks in this study.

Table 3.3 – Dynamic properties of rocks and minerals (GCTS, 1994)

Rock or Mineral	Apparent specific gravity	P wave velocity (m/s)
Siltstone	2.6	1610
Basalt	2.86	5400
Limestone	2.3 ~ 3.0	3200 ~ 5500
Quartz	2.65	5225
Marble	2.88	4950
Sand	1.4 ~ 2.0	300 ~ 1300
Sandstone	2.1 ~ 2.9	2000 ~ 3600
Shale	2.51 ~ 2.72	2250
Coal	1.2 ~ 1.5	1100 ~ 2800

3.3.6 Limitations

There are several limitations of this system, which will be highlighted in the following. The pulse may be significantly weakened when it reaches the receiving transducer as a result of it passing through randomly oriented discontinuities. Selecting transducers with a lower operating frequency can minimize these attenuation losses. Pulse amplitude may also reduce if the coupling between the transducer platens and specimen is weak. The application of an acoustic coupling material, e.g. honey or grease, can significantly improve the shear and compression wave amplitude.

The sample anisotropy can influence the test results as the waves may experience reflection and mode conversion as they pass through boundaries of materials with different velocity properties. Additional factors that can affect wave velocity propagation in rocks include rock type, texture, density, porosity, stress, water content and temperature.

Chapter 4: Jurong Formation

This chapter will cover a summary of the site investigations that had been conducted at Jurong Island, which is part of Jurong Formation, for its potential development of underground hydrocarbon storage caverns. Preliminary laboratory testing results of rock cores will be presented as well and further testing will be carried out in this study to investigate its engineering characteristics.

4.1 General

With the recent successful completion of Singapore's first underground rock cavern in Bukit Timah Granite at Mandai, researchers had since conducted extensive investigations on the sedimentary rocks of Jurong Formation. Some of the suitable sedimentary rocks to host underground caverns include limestone in Pandan and Tuas area, conglomerates and sandstone in NTU campus and Labrador Park. The site investigations conducted include geological mapping, seismic reflection surveys, deep coring and geophysical borehole logging as well as laboratory testing to assess its engineering properties and its potential for underground cavern development.

The general geological characteristics of Jurong Formation had been covered in the literature review (See section 2.4 – Jurong Formation). As such, the following section will discuss the engineering geology of Jurong Island.

4.2 Jurong Island

The Jurong Island is located southwest of the main Singapore Island, separated by Selat Jurong Strait. The site was originally an offshore area but it is now reclaimed with sand and covering approximately 11 km² (length of 3.5 km and width of 3.1 km). The existing ground level is at an average elevation of 5 m above the mean sea level.

4.3 Site Investigations

Site investigations had been carried out to examine the geological features of the subsurface ground, the hydro-geological situation of the site, and the general rock mass quality. These investigations included a total of 10 km of surface seismic reflection profiling survey, 8 exploratory boreholes to a depth of approximately 100 m and 2 deep exploratory boreholes to a depth of 200 m, yielding a total of 810 m in length of drilled cores. Downhole geophysical logging, various site and laboratory tests were conducted as well which will be elaborated in the following sections.

Fig 4.1 – Investigation site, borehole location and geophysical survey lines at Jurong Island

4.3.1 Surface Geophysical Survey

A total of 10 km of surface seismic reflection profiling survey was conducted along 6 survey lines intersecting deep drilling boreholes (Fig 4.1). They were undertaken using a geophone spacing of 5 m, along Lines 1, 2, 3 and 4 for deep investigation to a depth of 500 m.

The objectives of surface seismic reflection survey are as follows: -

- Determining the depth and profile of the superficial deposits and weathering zone;
- Examining the stratigraphic and structure of the sedimentary formations;
- Assessing the structural geology and locating the faults and weakness zones; and
- Providing data from different sources for correlation studies.

The acquired raw data are processed through filtering and correction. Finally, an interpreted geological profile was produced based on the reflection interface, together with borehole coring and downhole seismic logging data.

4.3.2 Borehole Drilling and Logging

Exploratory boreholes were drilled and logged to confirm the findings of the surface geophysical surveys and to evaluate the rock mass quality. A total of 10 boreholes were drilled where 8 of them (BH1, BH2, BH3, BH4, BH6, BH8, BH9 and BH10) were to a maximum depth of 100 m, and the remaining 2 (BH5 and BH7) to a maximum depth of 200 m below ground level. In general, the boreholes were spaced at a distance of between 800 to 1200 m, thus providing a good coverage of the site (Fig 4.1).

Conventional borehole logging included assessment of the consistency of the residual soil using Standard Penetration Tests (SPT), and the quality of the bedrock by Rock Quality Designation (RQD).

4.3.3 Downhole Geophysical Logging

Downhole geophysical logging was carried out during borehole drilling in conjunction with conventional logging. The main objectives were to measure the physical and mechanical properties of the rock mass, and conduct correlation studies with other investigation methods.

Three logging methods were used, including: -

- Caliper logging, for examining the changes in lithology of rocks encountered in borehole and locating fractures intersected by the borehole;
- Closed circuit television logging, for providing a visual picture of the borehole wall, and hence indicating the location, pattern and degree of fracturing of rock mass; and
- Seismic P-S waves logging, for evaluating the strength and deformation characteristics of rock mass from P-S wave velocity.

4.3.4 Field Groundwater Testing

Groundwater tests carried out at site include in situ rising/ falling head permeability test, standpipe and piezometer groundwater monitoring in the drilled borehole. With these tests, the groundwater table and pore water pressure in the area were observed and measured, and the rock mass permeability was determined.

4.4 Geology and Hydrogeology

The geological formation and structures of Jurong Island will be presented in the following section. The hydrogeology of the site will be discussed as well.

4.4.1 Geological Formations

Jurong Island is part of the Jurong Formation and its geology had been reviewed in section 2.4 (Chapter 2 – Literature Review).

The majority of the bedrock at the site belongs to Tengah Facies and Ayer Chawan Facies of the Jurong Formation, which is known to be laid down in shallow water during the Late Triassic times. It comprises sedimentary rocks, mainly marine mudstone, muddy siltstone, sandstone, conglomerate and limestone. Low grade of metamorphism is observed on these rocks. Due to the chemical decomposition under humid tropic climate, the rocks have been highly

weathered and often overlain by a thick layer of residual soil and completely weathered rock.

4.4.2 Geological Structures

In a previous study on the tectonic setting of the Jurong Formation, Redding (1999) has constructed their theory stating that the basin Jurong Formation has been uniformly uplifted with little tilting, and rock strata generally has not been deformed into major folds. Based on the evidence available from site investigation at Jurong Island – mainly from the bedding observed in the drilled cores and profiling obtained from seismic reflection survey – the original layering of the sedimentary rocks have not been strongly disturbed and there were no significant folding structures. Redding's theory seems to provide a reasonable geological model for the site geology.

However, there is still evidence of past structural tectonic activities across the site with the development of faulting, fracturing and metamorphism.

Faulting is observed by the brecciated and weakness zones of cores and zones of drilled core loss, as well as the uplifting of the rock layers interpreted in the seismic profiling. A series of faults, dipping at high angles of 60° to 80°, can be observed from the investigation results. These fault strike along two main directions – one from northwest to southeast direction and another from northeast to southeast direction.

Mudstone and siltstone are more prone to faulting due to its relatively weak nature. Another consequence of faulting is the intensive fracturing of rock mass along the fault planes, with a corresponding reduction in rock mass strength and quality.

4.4.3 Hydrogeology

Groundwater was found in all exploratory boreholes. Standpipes and piezometers were installed in the boreholes to monitor the groundwater table and pore water pressure. The monitoring indicated that the groundwater table is usually within the reclaimed sand, and often located at about 3.0 to 3.5 m below ground surface. The elevation of groundwater table varies between 0 to 3.5 m, and at an average of 2.0 m above mean sea level. The groundwater table is very consistent and stable across the site.

Chemical testing was carried out on 20 samples of groundwater and result shows that water is not aggressive. The groundwater temperature is stable around 24°C.

4.5 Engineering Properties of Rocks

The overburden and degree of weathering of the sedimentary rocks will be elaborated. The rock types encountered and rock strength will be discussed as well.

4.5.1 Overburden and Weathering

The Jurong Formation has been subjected to intensive chemical weathering due to the humid tropical climate of Singapore. The sedimentary rocks of Jurong Island have been strongly weathered and overlain by a thick layer of residual soil. The thickness of residual soil and completely weathering zones ranges from 10 m to 46 m, with an average of 24 m.

The degree of weathering of the rocks is mainly governed by the rock type and fracturing, and can be classified according to the classification system suggested by Guo and Zhao (1998). In general, the weathering becomes less severe with the increase in depth; i.e. from completely weathered to fresh rocks. However, there maybe exception to encounter strongly or completely weathered rocks below fresh or slightly weathered rocks due to the existence of fracturing zones at depths.

4.5.2 Rock Types

A variety of rocks are found in the drilled boreholes. The common types are siltstone and sandstone, with the less common conglomerate, mudstone and limestone. Table 4.1 summarizes the variation of thickness of rock types encountered.

a) Siltstone

Siltstone is one of the major rock types of the Jurong Formation. It was encountered in all the ten boreholes. The total apparent thickness of siltstone encountered in various boreholes varies from 4.3 m (BH3) to 88.9 m (BH2), with the average thickness of 36.3 m. Individual siltstone horizons vary from 0.2 to 19.0 m thick, averaging 4.0 m thick.

Thick layers of siltstone are encountered in BH1, BH2, BH6, BH8 and BH10 at the northern and central parts of the site. The siltstone layer is relatively thin elsewhere.

b) Sandstone

Sandstone is also one of the major rock types of the Jurong Formation. It was also encountered in all the ten boreholes. The apparent thickness of sandstone layers observed in the boreholes varies from 6.0 to 138.0 m, with an average thickness of 30.6 m.

Thick layers of sandstone are encountered in BH3, BH4 and BH7 at the southern and central parts of the site. The sandstone layers are thin elsewhere.

c) Conglomerate

Although conglomerate is one of the major rock types of Jurong Formation, its distribution is limited at the Jurong Island site. It was encountered in BH4 and BH7A. The apparent thickness of conglomerate layers observed in the two boreholes is 4.5 and 22.0 m respectively. Individual conglomerate horizons vary from 0.6 to 9.0 m thick, averaging 2.1 m thick.

The conglomerate layer encountered in BH7A at the southern part of the site is thick, while at the eastern part, only a thin layer of 4.5 m is present. Conglomerate is not present elsewhere.

d) Mudstone

Mudstone is not widely distributed throughout Jurong Island compared to siltstone and sandstone. It was encountered only in BH1, BH3 and BH7, located at the western and central parts of the site.

The apparent thickness of mudstone observed in the above-mentioned boreholes varies from 2.3 to 5.8 m, with an average thickness of 2.7 m.

e) Limestone

Limestone of Jurong Island is of Late Triassic age and a part of Ayer Chawan Facies. It was encountered in BH7A and BH9 at the south-eastern and southern parts of the site. Limestone is not present elsewhere.

The apparent thickness of limestone observed in the two boreholes is 23.5 and 25.8 m respectively.

Table 4.1 – Summary of thickness of rock types encountered (Tan et al., 2002)

Borehole	Total thickness (m)				
	Siltstone	Sandstone	Conglomerate	Mudstone	Limestone
BH1	45.6	9.8	-	4.1	-
BH2	88.9	6.5	-	-	-
BH3	4.3	46.8	-	5.8	-
BH4	25.4	19.5	4.5	-	-
BH6	38.9	15.2	-	-	-
BH7A	-	113	25	2.3	23.5
BH8	52.9	14.3	-	-	-
BH9	8.2	13.0	-	-	25.8
BH10	39.5	12.3	-	-	-

4.5.3 Rock Strength

Jurong Town Corporation (JTC) had conducted several laboratory tests to determine some of the parameters. The laboratory tests indicate that the rock strength depends primarily on rock types and degree of rock weathering. Table 4.2 shows the various index properties of the rock types encountered. It should be noted that these results are obtained from limited test quantity and only can be used as an indication of the likely range of the rock properties.

Table 4.2 – Various index properties of rock types encountered (Tan et al., 2002)

Rock Type	I_{s50} (MPa)	σ_c (MPa)	E (GPa)	σ_t (MPa)	V_{swell} (%)
Siltstone	1.9 ~ 9.5	30.0 ~ 66.5	20.0 ~ 35.0	14.4 ~ 20.0	0.32 ~ 2.09
Sandstone	3.6 ~ 12.9	46.4 ~ 161.9	14.5 ~ 98.0	5.6 ~ 21.2	-
Conglomerate	4.7 ~ 8.1	31.1 ~ 62.5	23.5 ~ 46.1	6.8 ~ 11.2	0.44 ~ 0.91
Mudstone	2.3 ~ 3.0	-	-	4.2 ~ 10.9	0.72 ~ 1.86
Limestone	4.0 ~ 9.9	-	-	-	-
Note: I_{s50} = Point load strength index; σ_c = Uniaxial compressive strength; E = Young's modulus; σ_t = Brazilian tensile strength; V_{swell} = Volume swelling index.					

Based on the testing results of point load strength index and uniaxial compression strength, the rocks can be classified into different categories in accordance to BS 5930: 1999. *Siltstone* can be classified as a moderately strong to strong rock, *sandstone* as a moderately strong to very strong rock, *conglomerate* as a moderately strong to strong rock, *mudstone* as a moderately strong, and *limestone* as a strong to very strong rock.

4.5.4 Discontinuities in Rock Mass

Discontinuities in rock mass include fault zones and rock joints, which are inevitable due to tectonic activities. Based on the exploratory borehole logs together with borehole video logging and surface seismic surveys, it is estimated

that there are 38 fault zones, with dip angles varying between 55° and 75°. These fault zones represent 13.3 % of the total rock cores from the exploratory boreholes. The width of the fault zones varies between 0.2 and 16.8 m, averaging 2.1 m. This highlight an important feature that the fault zones will lead to the loss of rock cores in borehole coring and the very low RQD values of rock mass. Table 4.3 shows the distribution of fault zones in exploratory boreholes.

Table 4.3 – Distribution of fault zones in exploratory boreholes (Tan et al., 2002)

Borehole	Total length of fault zone (m)	Percentage of fault zone in borehole (%)
BH1	38.6	62.5 %
BH2	8.8	9.3 %
BH3	16.6	27.7 %
BH4	2.6	5.0 %
BH5	13.1	10.5 %
BH6	4.3	7.2 %
BH7A	5.8	3.5 %
BH8	13.9	20.7 %
BH9	0.6	1.2 %
BH10	0.6	1.1 %

The most prevalent joints are steeply inclined sub-vertical 70° to 85°, moderately to widely spaced joints, generally within the strong and moderately strong rocks such as sandstone, conglomerate and siltstone. They are formed mainly due to local faulting and can be described as smooth and planar or rough and planar joints.

Other joint sets are inclined shear joints with dip angle between 40° and 65°. The joints are closely spaced in fault zones and within highly weathered rocks, while those in slightly weathered to fresh rocks at greater depths are moderately spaced. Moreover, these joints are sometimes filled with 1 to 3 mm of clay.

Based on the borehole core logs, the fracturing of rock mass due to the faults and joints can be assessed using the standard *Rock Quality Designation* (RQD), defined as:

$$RQD = \frac{\sum \text{length of cores} \geq 10 \text{ cm}}{\text{drill length}} \times 100 \% \tag{4.1}$$

Table 4.5 shows a range of RQD values with the corresponding description of the rock mass quality.

Table 4.4 – RQD values and description of rock mass quality (Bieniawski, 1989)

RQD (%)	Description of Rock Mass Quality
90 ~ 100	Excellent
75 ~ 90	Good
50 ~ 75	Fair
25 ~ 50	Poor
0 ~ 25	Very poor

An example of RQD values logged in one of the boreholes is shown in Fig 4.2 (See Appendix A4.1 for other boreholes). In general, the RQD values increase with depths; however, there are exceptions whereby the reverse is the case. The RQD values range from 20 to 100 with a weighted average value of 60.

It should be noted that RQD only accounts for the degree of fracturing of rock mass, rather than the direct measure of the overall rock mass quality. The rock mass quality (Q) is a combination of rock material and rock fracture properties, including the degree of fracturing (See section 2.3.4 – Rock Mass Quality of Bukit Timah Granite).

Fig 4.2 – Example of RQD and Q-values of rock mass in one the of the boreholes (BH 2) (Tan et al., 2002)

4.5.5 Rock Mass Permeability

The degree of fracturing will determine the permeability of the rock mass. In laboratory testing, the rock mass permeability was measured by the rising/falling head permeability tests. The results of the permeability tests varied from 10^{-7} to 10^{-6} m/s, which can be classified as moderately permeable.

4.6 Rock Mass Quality Assessments

Rock mass quality can be used to assess the site ground conditions with regard to the suitability of rock cavern construction. In addition, the rock mass quality will govern the rock cavern design and affect the excavation cost, which will be reflected in the amount of temporary and permanent rock support required during construction.

The rock types and characteristics can be obtained through the core logs and laboratory testing. The characteristics of rock joints/ faults and rock mass encountered at the Jurong Island site are essential to study the feasibility of cavern development.

4.6.1 Rock Mass Classification Q-System

The Q-system had been adopted to evaluate the rock mass quality of the sedimentary rocks. The details of the evaluation system had been covered in section 2.3.4 (Chapter 2 – Literature Review).

4.6.2 Rock Mass Assessment for the Site

The assessments of the six parameters and the estimation of Q-value were carried out for the entire cores in each of the boreholes. An example of the estimated Q-values along one of the boreholes is shown in Fig 4.2, in 5 m length intervals of rock cores along the depth.

The *RQD* values are derived from the core logs and they are generally within the range of 20 to 100.

The *J_n* values are obtained based on the assessment of the number of discontinuity sets that are present over each 5 m length of cores. In this site, there are two to three joint sets down to an elevation of approximately 60 m, beyond which, the number is reduced to two and finally one set at greater depth. The assessed values of *J_n* generally range from 2 to 20.

The *J_r* values are estimated according to the qualitative description of the roughness and planarity of the joints, over each 5 m length of cores observed in the core logs. If more than one joint set exist within the length of cores, the smoothest set will be used in the estimation of *J_r* value. The estimated values of *J_r* generally range from 1.5 to 3.0.

The *J_a* values are assessed from the qualitative description of the degree of alteration that the joints surfaces have undergone, over each 5 m length of cores. Similar to the *J_r* parameter, the most altered joint set should be used if more than one set exist within the length of cores. The assessed values of *J_a* are generally within the range of 1.0 to 3.0.

The J_w values have been assigned based on the estimated amount of groundwater inflow during excavation. A conservative value of 0.66 for J_w has been generally adopted, indicating medium inflow or occasional outwash of joint fillings. This is an assessment based on experience from hydrological conditions compared with the Jurong Island site.

The SRF values have been assessed with a systematic approach. For shallow depths of less than 50 m where the rock mass is generally competent, an SRF value of 2.5 is applied to reflect the effect of low stress. However, if discrete fault zones exist in the competent rock mass, an SRF value of 5.0 is applied to reflect both the effect of low stress and loosening of rock mass at the fault zone. When the depths are greater than 50 m, an SRF value of 1.0 is assigned to the competent rock mass to reflect a medium stress condition. An SRF value of 2.5 is adopted for the discrete fault zone to reflect a slight loosening of rock mass.

The overall assessment of the rock mass quality (Fig 4.3) reveals that about 40% of the total logged cores from the 10 boreholes are classified as *good* ($Q = 10 \sim 40$) to *extremely good* ($Q = 100 \sim 400$) quality rock mass and 15% of it as *fair* ($Q = 4 \sim 10$). On the other hand, about 45% are classified as *exceptionally poor* ($Q < 0.01$) to *poor* ($Q = 1 \sim 4$) quality rock mass.

Fig 4.3 – Overall assessment of rock mass quality (Tan et al., 2002)

Chapter 5: Experimental Setup, Testing Program & Sample Preparation

This chapter will give a detailed description of the experimental set up of the uniaxial and triaxial compression tests, testing standards, and procedures. The capacity and calibration of displacement and pressure transducers are presented. In addition, the set up of acoustic emission detecting device and ultrasonic testing equipment are also discussed.

The testing program, which includes physical property tests, compression tests, tensile tests and petrographical analyses, will be presented. Finally, the sample location, selection and preparation are detailed.

Fig 5.1 – An overview of the rock triaxial compression testing system

5.1 Rock Triaxial Testing System

Fig 5.1 shows an overview of the apparatus used to perform the compression tests. This servo-controlled triaxial testing system consists of three components: compression machine, servo-hydraulic system and data acquisition system. A schematic diagram showing the testing system is illustrated in Fig 5.2, which shows the closed loop control of feedback signal system. The signal conditioning, computer interface box, servo amplifier and switch control box have been integrated as a single unit by *Instron Labtronic Controller 8800*. The testing system continuously compares the demand signal from the controlling software with the feedback signal from the controlling transducer. If there is a difference in the signals, the comparator in PID controller generates an error signal to drive the actuator in the direction necessary to reduce the error to zero.

Fig 5.2 – Schematic diagram of the rock triaxial testing system (GCTS, 1992)

5.1.1 Compression Machine

The model of the compression machine is *Stiff Test System STS-1000*, which was manufactured by Structural Behaviour Engineering Laboratories, Inc. (SBEL) in United States. It is designed for static and dynamic closed-loop operation and is capable of performing strain or stress controlled as well as post failure behaviour type test. It comprises a four-columns testing frame, load cell, servo controlled hydraulic actuator and triaxial cell as shown in Fig 5.3 and 5.4 respectively.

Fig 5.3 – Testing load frame and load cell (All dimensions are in mm)

5.1.1.1 Loading Frame

The testing frame consists of four columns with an overall column height of 1.5m and a square base area of 0.93 x 0.93 m with a thickness of 0.175 m. Upper crosshead is adjustable with an allowance of 0.5 m with threaded columns. The stiffness of the testing frame is 5.25 MN/mm. The capacity of the loading frame is 600,000 lb (2669 kN).

5.1.1.2 Triaxial Cell

The *SBEL Rockcell Model 10* triaxial cell is designed for NX core size of 54 mm diameter and has a maximum confining pressure capacity of 138 MPa.

A removable NX core bottom platen with O-ring grooves for sealing the specimen jacket is threaded on top of the 150 mm diameter base plate as shown in Fig 5.4. A loading piston with spherical seat is designed to place on top of the core samples to minimize the stress concentrations due to non-parallel specimen ends. Top triaxial cell (cell sleeve) has a wall thickness of 30 mm and an outer diameter of 215 mm. Eight 38 mm diameter tie-rods with nuts tighten up cell sleeve and base plate (Fig 5.4). All components are constructed of stainless steel. The configuration of the fusites connecting to measuring devices on the base plate is detailed in Fig 5.5.

Fig 5.4 – Cell pressure chamber

Fig 5.5 – Section A-A view on the configuration of fusites on triaxial base plate

5.1.1.3 Pressure Intensifier

The pressure intensifier, manufactured by SBEL, has a capacity of 70 MPa with 1,000cc stroke capacity. It is designed for closed-loop digital servo control of pressure (Fig 5.6).

Fig 5.6 – Pressure intensifier

5.1.2 Servo-Hydraulic System

The second component of the rock triaxial testing system is the servo-hydraulic system. Fig 5.2 shows the integration of SBEL compression machines and Instron digital servo controller and operating software to perform the computer controlled triaxial tests. Hydraulic actuator of compression machine and intensifier for confining pressure are operated by the command signal sent out by *Instron Labtronic 8800* controller to the solenoid valve of hydraulic pump. The pressure capacity of servo valve is 69 MPa, which enables actuator to be operated at the maximum load of 1100 kN.

5.1.2.1 Hydraulic Pump

Fig 5.7 shows the complete operating hydraulic flow system of the hydraulic pump. Hydraulic oil is powered by AC Lincoln Motors with 30 horsepower and 1760 RPM, which is operated by a NEMA voltage starter. A flow control valve is used to monitor the consistency of hydraulic flow rate of the pump. The hydraulic pump is equipped with an ACA hydraulic accumulator. Two fluid filters are used for filtering outgoing and incoming hydraulic oil in and out of hydraulic oil reservoir. A water-cooled industrial oil cooler of Thermal Transfer Products, Ltd., is used to cool down the oil temperature.

Fig 5.7 – Complete hydraulic flow of the rock triaxial testing system

5.1.2.2 Instron Labtronic 8800 Computer Interface System

The 8800 Control System incorporates a standard computer interface as a mean of system control. It allows full control of the test instrument and associated devices, and enables recording and processing of system test data, by a Pentium II 366 kHz computer. The Labtronic 8800 as shown in Fig 5.8 is a compact tower providing up to six channel rack module, which can control up to four independent tests simultaneously in multi-station mode.

The computer interface is based on the electrical, cabling, and timing specifications of the HS488 standard for a General Purpose Interface Bus (GPIB) to communicate between the controller and the computer (Instron, 1999). This allows the interconnection of two or more devices, and is implemented using the IEEE 728 standard for message code and format conventions.

Fig 5.8 – Instron Labtronic 8800

5.1.2.3 Instron Fast Track 8800 Material Test Control System

The *Instron Fast Track 8800* material test control system is a fully digital control system and designed as an integral part of the Instron Fast Track range of materials testing machines, controllers and software.

The *Fast Track 8800* material test control system is a family of modular controllers. The controller is configured for single and biaxial testing. Each axis within control system consists of a separate Integrated Axis Controller (IAC) board. To provide the feedback signals required by the controller, and also to give the ability to acquire data from compression machine, each IAC can have up to four Sensor Conditioner Modules (SCM) mounted on it. The outputs from the SCMs can be read directly by the control computer.

Advanced fatigue test software application *Wavemaker* is used to perform the compression tests. The capabilities of *WavemakerEditor* include block loading with a variety of mode changes, absolute or relative ramps, sines and other wave shapes as shown in Fig 5.9. Data can be acquired continuously or periodically throughout the test sequence. Up to five real time graphs may be displayed

Fig 5.9 – WavemakerEditor for programming different modes of testing

5.1.3 Data Acquisition System

The third component of the triaxial testing system is the data acquisition system. The *Instron WavemakerRunner* controls the internal data logging system whereas the TML Static Measurement Software, which is used for uniaxial compression test, controls the external data logging system.

5.1.3.1 Internal Data Logging System

The 8800 data logging system is fully integrated with all of the internal event detectors, and is synchronized with the internal data acquisition clock. This means that at any event detector in the system is capable of automatically starting or stopping the data logging process within one millisecond of the time that the event is detected. Furthermore, because all data logging runs synchronously with respect to the internal sampling clock,

the data logged into the buffers is an exact subset (depending upon specified logging interval) of that used to close the servo control loop of the testing machine. This means that by analysing a buffer of logged data, one can learn exactly what the system response is to any given stimulus as shown in Fig 5.10.

When the data snapshots are stored in order of occurrence in internal buffers until enough of them have accumulated to meet the buffer size criterion specified by the user, the data can then be transferred from the buffer to an external computer *Instron WavemakerRunner* as an ASCII file in Command Separated Value (CSV) format.

Fig 5.10 – Real time display of measuring devices

5.1.3.2 External Data Logging System

An external 10-channel *TML Data Logger Model TDS 302* (RS-232C interface) is used for recording data from strain gauges and load cell. Visual LOG-Static Measurement Software is the data logging software for Windows 95 with the capability of automatic data acquisition in the minimum interval measurement of one second. Recorded data can be filed and converted into CSV format for utility on MS Excel. Fig 5.11 shows the block diagram of the external data logging system.

**Fig 5.11 – External data logging system;
(a) Block diagram, (b) Data logger TDS 302**

5.2 Displacement and Pressure Measuring Devices

The displacement and pressure measuring devices used in the experiments will be presented in the following sections.

5.2.1 Linear Variable Deformation Transducer (LVDT)

The *Solartron* LVDT (± 75 mm) is mounted on the compression machine as a position control device for the Instron Fast Track 8800 (Fig 5.12). Its accuracy is up to one thousandth of a millimetre (0.001 mm).

*Fig 5.12 – (a) Vertical displacement LVDT mounted on column,
(b) Magnetic stand on bottom platen*

5.2.2 Electromagnetic Transducer

The electromagnetic LVDT is a product of *Schaevitz*. The AC-operated MHR series of LVDT has a frictionless operation, which gives infinite resolution. It is an electromechanical device that produces an electrical output proportional to the displacement of a separate movable core. It consists of a primary coil and two secondary coils symmetrically spaced on a cylindrical form as shown in Fig 5.13.

A free-moving, rod-shaped magnetic core inside the coil assembly provides a path for the magnetic flux linking the coils. When the primary coil is energized by an external AC source, voltages are induced in the two secondary coils. These are connected series opposing so the two voltages are of opposite polarity. Therefore, the net output of the transducer is the difference between these voltages, which is zero when the core is at the centre or null position. A differential voltage is induced and changes linearly with rod position when the rod is moving away from the null position (Fig 5.13).

Fig 5.13 – Displacement measuring principle of electromagnetic LVDT

The SBEL axial and circumferential deformation device shown in Fig 5.14 is used to measure the average change in specimen height and diameter of rock cores in uniaxial/ triaxial compression test.

Fig 5.14 – SBEL axial and circumferential deformation device

As shown Fig 5.14 and 5.15, two LVDTs model 250MHR (labelled as 4) are used for axial displacement measurement over a nominal linear range of ± 6.35 mm. In addition, one LVDT model 100MHR (labelled as 5) is used for circumferential displacement measurement over a nominal linear range of ± 2.54 mm.

Fig 5.15 – Electromagnetic LVDTs

A chain attached with a 100MHR LVDT, as shown in Fig 5.16, measures the circumferential deformation, which can be converted into lateral deformation easily.

Fig 5.16 – Section A-A of Fig 5.14; Circumferential chain (GCTS, 1992)

5.2.3 Strain Gauge

TML polyester strain gauges with a length of 20 mm and resistance of $120 \pm 0.3 \Omega$ are used to measure the strain rate. Each sample is attached with a total of 4 strain gauges, 2 longitudinally and 2 transversely to measure the axial and lateral strain, respectively. The strain gauges are glued onto the pre-prepared surface (sanding and cleaning with solvent) with CN adhesive.

Strain gauge is constructed by bonding a fine photographically etched metallic resistance foil to an electrical insulation base using CN adhesive and attaching gauge lead. The strain generated in the specimen is relayed through the base to the fine foil where expansion or contraction occurs. As a result, the fine foil experiences a variation in resistance. This small variation is exactly proportional to the strain and the *TML Data Logger Model TDS 302* (discussed in section 5.1.3.2) is able to convert it to voltage output.

Strain gauging is done only at the initial stage of the test to calibrate the LVDTs. Subsequently, all strain readings are obtained from the electromagnetic transducers.

5.2.4 Load Cell

The *NX-H-600* load cell has a maximum capacity of 3 MN. It has an overall length of 388 mm with a swivel platen of 150 mm diameter. It is specially designed for *SBEL CT-1000* loading frame with a threaded adapter hole at the top of load cell (Fig 5.17).

Fig 5.17 – NX-H-600 load cell

5.2.5 Cell Pressure Transducer

Fig 5.18 shows the *Viatron 118* pressure transducer, which is used to introduce and measure the confining pressure in triaxial compression test. It has a capacity of 120 MPa.

Fig 5.18 – Viatron 118 pressure transducer

5.3 Acoustic Emission Detecting System

The system adopted to investigate the acoustic emission (AE) of the rocks is the *AESMART 2000* from *DECI* (Dunegan Engineering Consultants Inc.) (Fig 5.19). The transducer and instrument in this system are designed to utilize modal analysis of the AE signal to separate valid signals from extraneous noise. (Refer to Appendix B5.1 for a detailed AE detecting system setup)

Fig 5.19 – AESMART 2000 acoustic emission detecting system

5.3.1 Acoustic Emission Piezoelectric Transducer - SE1000-HI

The function of the *SE1000-HI* transducer is to convert the mechanical energy associated with an acoustic emission event into a suitable electrical signal. The transducer is designed to operate as a displacement sensitive device in the 20 - 60 kHz range of frequencies, which responds to the low frequency flexure wave produced by out-of-plane (OOP) sources in the 20 – 60 kHz range of frequencies. It also operate as a velocity sensitive device at frequencies above 100 kHz which responds to the high frequency extensional waves produced by in-plane (IP) sources, and to the high frequency shear waves produced by both OOP and IP sources.

SE1000-HI is not a resonant transducer. It has a very flat frequency response between 20 kHz to 400 kHz as shown in calibration curve in Appendix B5.2. The SE1000-HI transducer contains a cone shaped piezoelectric crystal. A mass is attached at the larger side of the cone. This feature along with its small aperture provides excellent frequency response to the surface displacement of an acoustic emission stress wave. The SE1000-HI is a +15V DC supply sensor with an integral line drive amplifier to minimize cable losses over long distance and to provide 20 dB of gain. It has a dynamic range of 74 dB and maximum frequency detecting range of 400 kHz. The size of the sensor is 20 mm in diameter and 22 mm in height as shown in Fig 5.20. The SE1000-HI has the highest fidelity (flat frequency response) of any acoustic emission transducer in the market. Their acoustical isolation assures reliable data even in noisy environments.

Fig 5.20 – SE1000-HI transducers

5.3.2 Acoustic Emission Device – AESMART 2000

The *AESMART 2000* hardware (Fig 5.21) splits the signal from the transducer into two frequency bands as shown in Fig 3.11 of Chapter 3. These two frequency bands are:

- 1) the low frequency (LF) output, which is the result of amplification and 20 – 60 kHz band-pass filtering of the input signal; and
- 2) the high frequency (HF) output, which is the result of amplification and 100 kHz hi-pass filtering of the input signal. Signals of each of these two frequency bands are analogue peak detected and read by an A/D converter. The computer then calculates the HF/LF ratio of these peak voltages.

Fig 5.21 – Block diagram of data flow through the hardware of the system (single channel operation adopted in testing)

The A/D converter has a sampling rate of 20 MHz and 8 bits of resolutions. The HF and LF peak detected voltages and the trigger Sync TTL signal (which is a ring down count counter counting number of time of HF amplitudes surpassing preset threshold of a single event) are recorded. All these information are

transferred to a *National Instruments PCMCIA DAQ 1200* card for laptop computer analysis via a ribbon cable.

5.3.3 Acoustic Emission Software – DECI-1C

The AESMART 2000 software *DECI-1C* (Fig 5.22) is equipped with the capacity of inputting a threshold for background noise settings, a HF/LF ratio which is a filter that will allow only acoustic emission signals with ratios above this selected value to be accepted into the data base. A “Ring down counter” present in the instrument will measure the counts above threshold for the HF channel for those signals which pass the filter ratio.

The gains of the trigger and data channels have two settings, i.e. 40 and 60 dB respectively. Gain for the data channel can be set at 30, 40, 50, or 60 dB. The thresholds for the HF, LF, Trigger input and Event count counter can be set by entering a desire value. The threshold counter provides counts above threshold from the HF channel for all valid events. The software is written to produce a default HF/LF ratio of 60 and maximum reading of HF and LF amplitudes of 10000 mV.

Fig 5.22 – DECI-1C software interface

5.4 Ultrasonic Velocity Testing System

The *GCTS* (Geotechnical Consulting and Testing Systems) *Ultrasonic Velocity Software* is used to determine the shear (S wave) and compression (P wave) wave velocities and store the waveforms digitally. The system utilizes a computer board for high speed data acquisition and a sophisticated computer software package for data analysis, which is integrated within the *GCTS Software Environment*. The standard *GCTS* ultrasonic test system configuration is shown in Fig 5.23 (refer to Appendix B5.3 for a detailed P and S wave setup).

Fig 5.23 – Standard GCTS ultrasonic test configuration

5.4.1 Data Acquisition Board

The *GCTS Ultrasonic Velocity Software* utilizes a high speed acquisition board, which can digitize analogue signals at a maximum real-time sampling rate of 40 MHz.

5.4.2 GCTS Pulser/ Receiver

The *GCTS* ultrasonic pulser/ receiver is digitally controlled with a 3 dB bandwidth at 35 MHz. A host computer controls the pulser/ receiver that

generates an electrical pulse that is sent to an ultrasonic piezoelectric transducer. The transducer will then convert the electrical pulse into an ultrasonic pulse, which will travel through the test specimen. At the other end of the specimen, another transducer will convert the pulse back into electrical signals for amplification and filtering by the pulser/ receiver unit. The amplified and filtered receiver output is then acquired by the data acquisition card and viewed using the *GCTS* ultrasonic software.

5.4.3 Combination P and S Wave Transducers

The transducers or crystals are fabricated from a ceramic material that behaves as a piezoelectric material. The transducers change shape when a voltage has been applied to them creating “pulses” which travel through the specimen.

The transducers typically have an operating pulse frequency between 200 kHz and 1000 kHz. This operating frequency is determined by the transducer thickness (in the direction of pulse propagation). An example of a combination P and S wave transducer arrangement is shown in Fig 5.24.

Fig 5.24 – Combination P and S wave platen

5.4.4 GCTS Ultrasonic Velocity Software

The software package is equipped with some of the key features, which will be highlighted in the following section.

The “face to face” delay time is the wave travel time observed when the ultrasonic platens are placed against each other (no specimen). The “face to face” delay times for both P and S waves can be entered so that the P or S wave velocity for a test specimen can be corrected by subtracting the face to face from the specimen results while the program is being executed. (Fig 5.25) After the delay time is obtained, it is verified by using an aluminium specimen with a known ultrasonic velocity (refer to Appendix B5.4 for the determination of P and S wave “face to face” delay time).

Fig 5.25 – Execute screen display

The high and low frequency filters are selected based on the frequency of the peak spectrum. In addition a “Bandpass” mode filter is used to filter out the

unwanted noise, only allowing the frequency band within the filters to be displayed.

The test data that was acquired in the Ultrasonic program (in *.wav form) can be displayed using the PlotWave command. The data can be displayed as the original data, a frequency spectrum, or as filtered data as shown in Fig 5.26. The first arrival time of each wave is manually taken off from the plotted filtered data.

Fig 5.26 – Combination data display selection

5.5 Testing Program

In this research study, extensive laboratory tests are conducted on sedimentary rock cores recovered from boreholes during the site investigation in Jurong Island. Some of the laboratory tests include: -

- a) Absorption and specific gravity test;
- b) Uniaxial compression test;
- c) Triaxial compression test;
- d) Tensile (Brazilian) test; and
- e) Petrographical analysis

Two non-destructive tests, i.e. acoustic emission and ultrasonic testing, are conducted in conjunction with the compression tests to investigate the fracturing characteristics of the rocks. Triaxial tests are performed at five different confining pressures, namely 5, 10, 20, 30 and 40 MPa.

Petrographical analysis is done to study the physical fracturing degree and mineralogy of the samples. In addition, the physical properties of the rocks are determined. The testing program conducted is summarized in Table 5.1.

Table 5.1 – Testing program for Jurong Formation sedimentary rocks

Rock Type	Absorption and bulk specific gravity test	Uniaxial compression test (UCT)	Triaxial compression test (TCT)	Tensile test (Brazilian)	Petrographical analysis
Siltstone	6	12	24	18	1
Limestone	6	8	12	11	1
Sandstone	4	9	30	20	1
Mudstone	4	3	13	10	1

5.5.1 Physical Properties Test

Absorption, specific gravity and surface moisture of different rock types are determined in accordance with “Suggested Method for Determination of the Water Content of a Rock Sample”, Method 1, ISRM Committee on Laboratory Tests, Document No. 2, First Revision, December 1977.

5.5.2 Compression Test

Uniaxial test is done in accordance with the “Suggested Methods for Determining the Uniaxial Compressive Strength and Deformability of Rock Materials”, ISRM Committee on Laboratory Tests, September 1978. Whereas the triaxial test is done in accordance with the “Suggested Methods for Determining the Strength of Rock Materials in Triaxial Compression”, ISRM Committee on Laboratory Tests, Document No. 7, First Revision, December 1977.

5.5.2.1 Calibration of Measuring Devices

Before the commencement of the compression tests, the axial and circumferential deformation devices (LVDTs) are calibrated by comparing its readings with that of strain gauges. The calibration factors of 0.62 and 1.45 for axial and radial respectively are found to be suitable for the conversion of LVDTs’ readings to that of strain gauges. The results of the calibration of these devices are shown in Appendix B5.5.

5.5.2.2 Acoustic Emission Settings

The acoustic emission transducers could detect the ambient noise when the HF threshold value is set just lower than the ambient noise level. The background ambient noise is typically detected at 500mV with trigger and data channels set at 60 & 50dB respectively as shown in the oscilloscope Fig 5.27(a). In order to detect any crack-like signals, HF has to be set slightly higher than ambient noise level.

Different combinations of trigger and system gains with different threshold values were performed on uniaxial compression test. It was found that with the trigger and data gain levels set at 60dB and 50dB respectively, and HF threshold level of 500mV, will give satisfactory results. The peak amplitude of HF voltage would not exceed the detecting range of the system, i.e. 10000mV.

Meanwhile, simulations of in plane signals by pencil lead breaking are performed on the surface and at the mid-depth of a hole drilled at the mid-height of the specimen. The HF/LF ratios are all below unity for pencil lead breaking on the surface as shown in Fig 5.27(b). And the HF/LF ratios are above 1 for the pencil lead breaking in the hole of specimen as shown in Fig 5.27(c) in the oscilloscope.

In order to confirm friction and impact noises can be filtered, one can hit the testing frame with a wrench. The system could detect the low HF and high LF signal, which gives a low HF/LF ratio of less than 1 as shown in the oscilloscope Fig 5.27(d).

Possible friction between specimen and platen was simulated by putting some Ottawa sand on the platen and rubbing it with the specimen. The detected HF/LF ratios are all less than 1 as shown in the oscilloscope Fig 5.27(e).

Typical rock crack initiation signal could be detected during compression test where HF/LF ratio is greater than 1 as shown in Fig 5.27(f).

(a) Background noise

Fig 5.27 – Oscilloscopes showing different detected signals triggered at HF threshold of 500mV (Channel 1 is HF, channel 2 is LF and trigger & data gain values set at 60dB and 50dB respectively)

(b) Pencil breaking on surface of specimen

(c) Pencil breaking at the mid-depth of a hole drilled at the mid-height of the specimen

(d) Wrench knocking

Fig 5.27 (continued) – Oscilloscopes showing different detected signals triggered at HF threshold of 500mV

(e) Friction between sample and platen

(f) Rock crack initiation under compression

Fig 5.27 (continued) – Oscilloscopes showing different detected signals triggered at HF threshold of 500mV (Channel 1 is HF, channel 2 is LF and trigger & data gain values set at 60dB and 50dB respectively)

5.5.2.3 Ultrasonic Test Settings

The ultrasonic test settings for P and S waves are tabulated in Table 5.2.

Table 5.2 – Ultrasonic test settings for P and S waves

Wave type	Face to face delay (μ s)	Bandpass filter frequency		Type of filter
		LF (kHz)	HF (kHz)	
P wave	10.8	80	150	Butterworth
S wave	23.9	100	250	Butterworth

The bandpass filter frequency settings are selected based on the occurrence of the peak frequency spectrum. From Fig 5.28, it can be seen that the peak frequency occurs in the range of 80 to 150 kHz for P wave. Hence, a bandpass filter is set between these frequencies to record the required wave spectrum. The same calibration was done for S wave, and the peak occurs in the range of 100 to 250 kHz.

Fig 5.28 – Bandpass filter frequency setting

5.5.3 Tensile Test

The tensile strength of the rock is determined by Brazilian Test, which is carried out in accordance with the “Suggested Methods for Determining Tensile Strength of Rock Materials”, ISRM Committee on Laboratory Tests, Document No. 8, March 1977. The experimental setup is shown in Fig 5.29. Acoustic emission is monitored during the test as well.

Fig 5.29 – Experimental setup of tensile test

5.5.4 Petrographical Analysis

The mineralogy, mineral grain size and pre-existing crack pattern of the rock samples can be determined by petrographical analysis. In this study, X-Ray Fluorescence (XRF) is adopted to study the thin sections of the rock samples under cross polarised light (XPL) and plain polarised light (PPL). In addition, major and trace elements are also determined (Appendix C6.2).

5.6 Sample Preparation

The preparation of the sample is done in accordance to International Society of Rock Mechanics (ISRM) suggested methods, with an aspect ratio of 2.5 to 3.0 and 0.5 for compressive and tensile tests, respectively.

Selected sample cores are cut to the required length using the *Melkon Servocut M350* abrasive rock cutter (Fig 5.30a). Subsequently, both ends of the sample are

polished to the acceptable tolerance and perpendicularity with the *SEBL* auto grinder (Fig 5.30b).

Fig 5.30 – (a) Metkon Servocut M350 rock cutter; (b) SEBL auto grinder

5.6.1 Sample Types and Selection

The 4 types of sedimentary rock types tested are siltstone, limestone, sandstone and mudstone.

In order to maintain consistency throughout, the selection of samples for a particular test is important. The following criterion shall be fulfilled: -

- a) Specimen shall have approximately the same mineralogy percentages of constituent minerals;
- b) Specimen shall be intact rock and free of visible joints; and
- c) Specimen for a particular test (e.g. triaxial test at 5 MPa confining pressure) shall preferably be from the same core run, i.e. around the same depth.

Chapter 6: **Presentation of Results & Discussions of Siltstone**

In this chapter, laboratory results of the various tests performed on siltstone will be presented. Observations from the two non-destructive tests (i.e. AE and P&S) will also be included.

6.1 Physical Properties

The physical properties of siltstone are determined in accordance with ISRM standard. The results of the specific gravity, absorption (as a percentage of dry weight) and surface moisture are shown in Table 6.1. (Refer to Appendix C6.1 for the detailed computation)

Table 6.1 – Physical properties of siltstone

Specific gravity	Absorption (%)	Surface moisture (%)
2.816	0.181	0.0303

6.2 Petrographical Analysis of Thin Sections

The thin sections of siltstone of *BH2 CR49* are analysed in X-Ray Fluorescence, XRF. The following subsections describe the siltstone in a hand-specimen and under thin section.

Hand-specimen description

Colour: - Dark greyish with lamina
Visible minerals: - Nil
Grain size: - Extremely fine
Structures: - Massive

Thin-section description

Mineral contents

It is impossible to give a semi quantification of mineral content (modal %) based on point-counting of minerals as the grains are too small to identify.

However, under the magnification of the microscope, visible mineral include quartz, feldspar, carbonates and opaque materials.

Description of textures

The rock shows a lamina defined by change in grain size and colour (Fig 6.1). The dark layers are composed almost entirely of clay-sized materials, where the lighter layers mostly consist of sand sized carbonate and small amounts of quartz and feldspar. The completely dark minerals could be an iron-compound, as the content of silicon (*Si*) and aluminium (*Al*) (Appendix C6.2) are high in the matrix. However, this most likely will be silicates.

Fig 6.1 – Laminated siltstone under crossed polarized light (XPL)

Additional descriptions given by the borehole log are as follows: -

- a) Dark grey;
- b) fresh;
- c) *SILTSTONE*;
- d) strong,
- e) tight to widely spaced, good fracture;
- f) excellent RQD;
- g) with quartz vein (6 cm thick); and
- h) fracture dipping 45 & 90 degree to core axis.

6.3 Compression Test

The results presented in the following section are based on samples at room condition tested under uniaxial and triaxial conditions.

6.3.1 Stress-Strain Curve

The deformation behaviour of the rock is presented in a form of a stress-strain curve. Data sampling rate of up to 5 Hz is performed to achieve a smooth curve. (5 Hz means 5 data sets per second; each set consists of 3 readings, namely load, axial and lateral deformation.)

6.3.1.1 Axial and Lateral Strain Curves

The maximum axial, lateral and volumetric strain of siltstone under various confining pressures are tabulated in Table 6.2. Appendix C6.3 summarises the test data for siltstone samples.

It can be seen that there is an increasing trend between the maximum strains and confining pressure.

Table 6.2 – Summary of maximum axial, lateral and volumetric strain of siltstone

Confining pressure, σ_3 , MPa (No. of samples)	Maximum axial strain, $\epsilon_a(\text{max})$ %	Maximum lateral strain, $\epsilon_l(\text{max})$ %	Maximum volumetric strain $\epsilon_v(\text{max})$ %
0 (12)	0.227 (± 0.062)**	- 0.087 (± 0.033)	0.054 (± 0.026)
5 (4)*	0.282 (± 0.076)	- 0.117 (± 0.041)	0.053 (± 0.014)
10 (5)	0.318 (± 0.061)	- 0.131 (± 0.012)	0.085 (± 0.034)
20 (4)	0.357 (± 0.043)	- 0.137 (± 0.013)	0.100 (± 0.040)
30 (4)	0.394 (± 0.081)	- 0.156 (± 0.057)	0.120 (± 0.064)
40 (7)	0.407 (± 0.064)	- 0.161 (± 0.060)	0.156 (± 0.073)
Note: * $\sigma_3 = 5$ MPa, number of samples = 4; ** $\epsilon_a = 0.227\%$, standard deviation = 0.062%.			

The typical stress-strain curves at various confining pressures are illustrated in Figs 6.2 & 6.3. From the graphs, it is observed that the slope of the axial strain curves is almost similar, i.e. constant Young’s modulus, for various confining pressures. However, in the case of the lateral strain, the slope of the initial part of the curve decreases with confining pressure. This behaviour will be discussed in the next section on Young’s modulus and Poisson’s ratio.

In addition, there is a distinct non-linear zone at the beginning of the curves, especially in the case of lateral strain.

Fig 6.2 – Typical stress-axial strain curves of siltstone

Fig 6.3 – Typical stress-lateral strain curves of siltstone

6.3.1.2 Young’s Modulus and Poisson’s Ratio

The Young’s modulus (E) and Poisson’s ratio (ν) of the rock samples are computed between the stress threshold of crack closure (σ_{cc}) and crack initiation (σ_{ci}), as the rock is assumed to be in the linear elastic deformation zone. The determination of the various stress thresholds will be further elaborated in section 6.3.2. A summary of results is shown in Table 6.3 and graphically illustrated in Fig 6.4.

Table 6.3 – Young’s modulus and Poisson’s ratio of siltstone

Confining pressure, σ_3 (MPa)	Failure stress, σ_f (MPa)	Young’s modulus, E (GPa)	Poisson’s ratio, ν
0 (12)	119 (± 25)	58.2 (± 3.9)	0.344 (± 0.038)
5 (4)*	132 (± 25)**	51.1 (± 2.3)	0.351 (± 0.035)
10 (5)	149 (± 12)	50.5 (± 4.2)	0.364 (± 0.035)
20 (4)	157 (± 26)	51.3 (± 2.2)	0.366 (± 0.032)
30 (4)	173 (± 31)	50.5 (± 4.0)	0.387 (± 0.008)
40 (7)	184 (± 24)	53.0 (± 4.8)	0.398 (± 0.037)

Note: * $\sigma_3 = 5$ MPa, number of samples = 4;
** $\sigma_f = 132$ MPa, standard deviation = 25 MPa.
Value in parenthesis denotes standard deviation for σ_f , E and ν .

Fig 6.4 – Young’s modulus and Poisson’s ratio of siltstone

The change in Young's modulus seems to be independent of confining pressure. There is a decrease at $\sigma_3 = 5$ MPa, and thereafter, it stabilises to an average of 51 MPa. This trend is further reinforcing the observation of constant slope of axial strain curves of various confining pressures.

For the Poisson's ratio, there is an increasing in absolute value, with respect to confining pressure. This phenomenon is due to the interaction of the change in axial ($\Delta \varepsilon_a$) and lateral strain ($\Delta \varepsilon_l$).

Poisson's ratio (ν) is given by Eq. 6.1 (ISRM, 1977): -

$$\nu = - \frac{\text{slope of axial strain curve}}{\text{slope of lateral strain curve}} \quad (6.1)$$

Thus, a constant axial strain slope and decreasing lateral strain slope will result in an increasing Poisson's ratio (absolute value).

6.3.2 Stress Thresholds Determination

Brace et al. (1966) and Bieniawski (1967b) proposed that stress thresholds could be determined using the axial, lateral and volumetric strain curves. The crack volumetric strain curve can be used to determine the crack initiation (σ_{ci}) in particular as suggested by Martin and Chandler (1994). Subsequently, Eberhardt et al. (1998) used a moving point regression technique which uses a "sliding window" approach to move through an x, y data set, fitting a straight line over a user-defined interval. The slope at each point is calculated over the interval and recorded; the process is repeated at successive points.

The cracking thresholds of Jurong Formation sedimentary rocks are determined using the methods shown in Table 6.4. The following sections will present the results of siltstone samples.

Table 6.4 – Stress threshold determination methods based on stress-strain curve

Identifying methods	Crack closure, σ_{cc}	Crack initiation, σ_{ci}	Crack coalescence, σ_{cs}	Crack damage, σ_{cd}
Brace (1964), Brace et al. (1966) & Bieniawski (1967b)	Axial strain Lateral strain Volumetric strain	point of non-linear zone changes to linear zone - -	- - -	- - point of reversal
Martin and Chandler (1994)	Crack volumetric strain	dilation begins after crack volume unchanged during elastic deformation	-	-
Eberhardt et al. (1998)	Axial stiffness	point shifting from incrementally increasing value to constant values	-	-
	Volumetric stiffness	-	starting of larger irregularities	stiffness values changed from positive to negative

Note: ‘-’ denotes not applicable.

6.3.2.1 Crack Closure (σ_{cc})

As defined by Brace et al. (1966) and Bieniawski (1967b), the stress threshold of crack closure (σ_{cc}) is the stress corresponding to where the axial strain starts to show linearity. However, it is not very obvious from the stress-strain curve of siltstone as it shows very marginal non-linearity during the closure of cracks. Using the moving point regression technique on siltstone sample 2-5-111.93 (typical sample) as shown in Fig 6.5, the axial stiffness increases slightly at the beginning of the test and reaches a near flat zone at a stress of **16 MPa**. This is the point where axial stiffness enters a near elastic region and it is defined as the stress threshold of crack closure (σ_{cc}).

Fig 6.5 – Identification of stress threshold of crack closure using axial stiffness

6.3.2.2 Crack initiation (σ_{ci})

Brace et al. (1966) and Bieniawski (1967b) defined the stress threshold of crack initiation (σ_{ci}) as the stress corresponding to the point where the volumetric and lateral strain curves departs from linearity, respectively. The crack initiation of siltstone (typical sample 2-50-111.93, $\sigma_3 = 20$ MPa) is approximately **41 MPa** as shown in Fig 6.6a.

However, the identification of this point is difficult especially in highly weathered samples. Martin (1993) suggested using the calculated crack volumetric strain to identify crack initiation. He defines crack initiation as the stress level at which dilation (crack volume decrease) begins in the crack volume after closure of cracks. Using Martin’s method, the crack initiation of sample 2-50-111.93 is **40 MPa** (Fig 6.6b).

Fig 6.6 – (a) Identification of stress thresholds using volumetric & lateral strains

Fig 6.7 –(b) Identification of stress thresholds using crack volumetric strain

Eberhardt et al. (1998) suggested that the crack initiation could be determined by identifying the stress level at the start of the relative irregularities region in the volumetric stiffness curve. However, this method is not applicable on this sample as shown in Fig 6.7.

Fig 6.8 – Identification of stress thresholds using volumetric stiffness

6.3.2.3 Crack Coalescence (σ_{cs})

The stress threshold of crack coalescence reflects the point whereby the volumetric stiffness curve starts to show large irregularities (Eberhardt et al., 1998). From Fig 6.7, σ_{cs} is found to be **100 MPa**.

6.3.2.4 Crack Damage (σ_{cd})

Crack damage stress threshold (σ_{cd}) is determined at the point of reversal of the volumetric strain curve (Bieniawski, 1967b). From Fig 6.6a, σ_{cd} is found to be about **58 MPa**. However, this phenomenon is not observed in majority of the siltstone samples.

Eberhardt et al. (1998) suggested that σ_{cd} can be identified from the volumetric stiffness whereby the stiffness value changes from positive to negative (Table 6.4). From Fig 6.7, σ_{cd} of sample 2-50-111.93 is about **102.6 MPa** and this differs from that determined from the volumetric strain curve method. The latter method is found to be more representative of the true crack damage from acoustic emission test, which will be discussed in a later section.

6.3.2.5 Failure Strength

The failure strength is taken as the peak stress the sample has been subjected to. Sample 2-50-111.93 has a failure strength of **110 MPa** (Fig 6.6).

6.3.3 Stress Threshold Results

A summary of the various stress thresholds is shown in Table 6.5 and Fig 6.8.

Table 6.5 – Stress thresholds of siltstone

Confining Pressure, σ_3 (MPa)		Crack closure, σ_{cc} (MPa)	Crack initiation, σ_{ci} (MPa)	Crack coalescence, σ_{cs} (MPa)	Crack damage, σ_{cd} (MPa)	Failure, σ_{lf} (MPa)
0 (12)	σ	24 (± 3)	54 (± 6)	80 (± 13)	114 (± 21)	119 (± 25)
	σ / σ_f	0.20	0.45	0.67	0.96	1.00
5* (4)	σ	16 (± 4)**	45 (± 9)	76 (± 16)	104 (± 19)	132 (± 35)
	σ / σ_f	0.12	0.34	0.58	0.79	1.00
10 (5)	σ	23 (± 1)	52 (± 5)	84 (± 17)	131 (± 34)	149 (± 12)
	σ / σ_f	0.15	0.35	0.57	0.88	1.00
20 (4)	σ	26 (± 2)	52 (± 2)	84 (± 20)	128 (± 31)	157 (± 26)
	σ / σ_f	0.17	0.33	0.53	0.81	1.00
30 (4)	σ	41 (± 3)	82 (± 5)	108 (± 23)	169 (± 37)	173 (± 31)
	σ / σ_f	0.24	0.47	0.62	0.97	1.00
40 (7)	σ	43 (± 6)	86 (± 8)	122 (± 24)	182 (± 24)	184 (± 24)
	σ / σ_f	0.23	0.47	0.66	0.99	1.00
Note: * $\sigma_3 = 5$ MPa, number of samples = 4; ** $\sigma_{cc} = 16$ MPa, standard deviation = 4 MPa. Value in parenthesis denotes standard deviation for stress thresholds.						

The stress threshold of crack closure, initiation and coalescence average 0.19, 0.40 and 0.61 of the failure strength, respectively. One point to highlight is the stress threshold of crack damage which averages 0.90 of the failure strength implies that crack damage only occurs prior to failure. This observation can be reinforced by the acoustic emission monitoring which will be discussed in a later section.

Fig 6.9 – Stress thresholds of siltstone

6.3.3.1 Failure Criterion

The Hoek-Brown failure criterion is an empirical criterion developed for application in underground excavation design. The experimental results are fitted using the following criterion (Hoek et al., 1995):

$$\frac{\sigma_1}{\sigma_c} = \frac{\sigma_3}{\sigma_c} + \left(m \frac{\sigma_3}{\sigma_c} + s \right)^d \tag{6.2}$$

where σ_1 = major principal stress;
 σ_3 = minor principal stress;

σ_c = uniaxial compressive strength of intact rock material;
 m , s and a are parameters that depend on the characteristics of the rock mass. For reasonable quality rock mass, the value of the parameter a is equal to 0.5, and the parameter s is equal to 1.0 for intact rock.

Fig 6.10 – Hoek-Brown failure criterion of siltstone

Fig 6.9 shows a plot of the Hoek-Brown failure criterion established from the experimental results. An m value of **4** (correlation = 0.55) with a uniaxial compressive strength (σ_c) of **119 MPa** is able to fit the Hoek-Brown failure envelope to the experimental results. The m value is obtained by the best fit method and verified by the *ROCKDATA Version 2.21* software.

The *ROCKDATA Version 2.21* software developed by Rock Engineering Group, University of Toronto is used to obtain the parameters for the Mohr-Coulomb failure criterion. The friction angle (ϕ) determined is **26.1°** and the cohesion (c) is **38.3 MPa**, i.e. $\tau = 38.3 + \sigma_n \tan 26.1^\circ$. The coefficient of

friction (μ) = $\tan 26.1^\circ = 0.49$. The program display of the software is shown in Fig 6.10.

Fig 6.11 – Mohr-Coulomb failure criterion of siltstone

6.3.4 Acoustic Emission Tests

The findings of AE tests which were carried out concurrently with compression tests are presented in the following section.

6.3.4.1 Acoustic Emission Counts

The detected signals of crack initiations and propagations are filtered into high and low frequency channels. The amplitudes of high frequency channel are detected and those surpassing the background threshold set at 500 mV are registered as ring down counts. The cumulative acoustic emission ring down count is plotted against stress as shown in Fig 6.11.

Fig 6.12 – Graph of cumulative emission ring down counts versus stress

For sample 2-49-108.80, the stress threshold of crack closure (σ_{cc}) of 30 MPa agrees well with that identified using lateral strain method suggested by Bieniawski (1967b) (see Table 6.4). However, the AE approach could not be used for the stress threshold of crack initiation (σ_{ci}) and crack damage (σ_{cd}) identification in this illustrated example.

It is observed that AE tests on siltstone are relatively ineffective in identifying the various stress thresholds. The graph shown in Fig 6.11 is the only one test that reflects a reasonable AE signature for siltstone. The rest of the tests only register AE counts towards the end of the test. It can be concluded that it is due to the behaviour of siltstone whereby the sudden propagation of cracks occur prior to failure.

6.3.5 Ultrasonic Tests

In addition to AE tests, the ultrasonic tests are conducted to investigate the behaviour of P&S wave velocities during compression tests.

6.3.5.1 Compression (P) Wave Velocity

A graph of P wave velocity versus axial stress is shown in Fig 6.12 for triaxial test at 20 MPa. In addition, the crack volumetric strain is plotted together to illustrate the relationship between the two properties. It should be noted that this sample is not the same as that previously discussed for the typical tests, as it is not possible to carry out both AE and ultrasonic tests concurrently on the existing experimental setup.

Fig 6.13 – P wave velocity and crack volumetric strain versus stress (2-50-111.93)

From the above figure, it is observed that both the P wave velocity and crack volumetric strain have a similar trend. This trend can be explained by the variation of crack volumetric strain (ϵ_{Vcrack}), whereby increasing ϵ_{Vcrack} means the sample is contracting, while decreasing ϵ_{Vcrack} means it is dilating. Hence, in the above illustrated example, the sample is contracting (compacted) during the initial stage till 15 MPa (σ_{cc}). Then, ϵ_{Vcrack} is more or

less constant which represents the linear elastic stage till 42 MPa (σ_{ci}). Subsequently, ϵ_{Vcrack} decreases with a sharp drop at 102 MPa (σ_{cd}).

The various stress thresholds determined using the stress-strain techniques are as follows: - $\sigma_{cc} = 16$ MPa; $\sigma_{ci} = 41$ MPa and $\sigma_{cd} = 103$ MPa. Therefore, it can be seen that the stress thresholds agree well for both ultrasonic and stress-strain methods. Further tests conducted at uniaxial and triaxial test at various confining pressures show similar findings.

6.3.5.2 Shear (S) Wave Velocity

Similar to P wave tests, S wave travel times through a sample are taken at regular load interval during a compression test. A typical plot of S wave velocity versus axial stress is shown in Fig 6.13.

Fig 6.14 – S wave velocity and crack volumetric strain versus stress

In Fig 6.13, the crack volumetric strain (ϵ_{Vcrack}) is plotted against stress as well to illustrate the inter-relation between the former and S wave velocity. Again, the similar trend of ϵ_{Vcrack} and wave velocity is clearly evident, as in the case of P wave. The stress threshold of crack closure (σ_{cc}) can be

identified at the point where the wave velocity is relatively constant ($\sigma_{cc} = 25$ MPa). At the stress threshold of crack initiation ($\sigma_{ci} = 56$ MPa), there is a sign of sample dilation (decreasing $\varepsilon_{V_{crack}}$), as well as a decrease in velocity.

The stress threshold of crack damage (σ_{cd}) is identified as the point of change in slope of the wave velocity curve ($\sigma_{cd} = 133$ MPa).

The various stress thresholds determined using the stress-strain techniques are as follows: - $\sigma_{cc} = 27$ MPa; $\sigma_{ci} = 54$ MPa and $\sigma_{cd} = 134$ MPa. Therefore, it can be seen that the stress thresholds agree well for both ultrasonic and stress-strain methods. Further tests conducted at uniaxial and triaxial test at various confining pressures show similar findings.

The stress thresholds determined in the S-wave method are higher than that in the P-wave method. This can be attributed to the higher strength of the S-wave sample as it is from a deeper depth (114.46m cf. 111.93m).

6.3.6 Failure Mode

There are basically two main types of failure mode observed in uniaxial and triaxial compression tests, namely slabbing and shearing. In uniaxial tests, failure occurs with development of one or more major cracks parallel to the direction of the applied force, resulting in a series of columns (Fig 6.14). This type of failure has been termed *slabbing* (Fairhurst and Cook, 1966) and it occurs when end restraint is eliminated.

Fig 6.15 – Slabbing failure of siltstone sample 2-49-111.40 ($\sigma_3 = 0$ MPa)

In triaxial tests, *shearing* of the test sample along a single oblique plane is observed. The angle of failure plane, which is measured from the horizontal, decreases as the confining pressure increases. In addition, the failure plane is almost perpendicular to the bedding planes (Fig 6.15).

Fig 6.16 – Shearing failure of triaxial samples

6.4 Tensile Test

The average tensile strength of siltstone is **15 MPa** (± 3 MPa) based one 18 samples tested in Brazilian Test as shown in Appendix C6.4. A typical stress-strain curve of a Brazilian Test on a siltstone is shown in Fig 6.16.

Fig 6.17 – Brazilian Test on siltstone

A plot of summation of counts versus tensile stress is shown in Fig 6.17.

Fig 6.18 – Acoustic emission test on siltstone under tensile stress

The stress corresponding to the point of departure from non-linearity to linearity is the stress threshold of crack initiation (σ_{ci}) (Fig 6.16). The initial non-linear part of the curve represents the crack closure stage. For sample 2-49-109.32, σ_{ci} is **7.7 MPa** ($0.41 \sigma_f$) and the rest of the samples have an average value of **0.43 σ_f** .

From Fig 6.17, the start of AE counts occurs at a tensile stress of **7.7 MPa**, which coincides with the stress threshold of crack initiation (σ_{ci}) determined in Fig 6.16. Thus it can be concluded that in this case, when σ_{ci} is reached, AE should start to occur. But the stress threshold of crack damage (σ_{cd}) could not be determined as there is no distinct feature in the AE versus stress plot.

The above illustrated result is only one of the few samples to produce a reasonable AE signature. The remaining tests show that there are little or no AE counts recorded throughout the Brazilian test. This is due to the inherent property of siltstone which is similar to the AE monitoring in compressive tests.

An example of a failed sample is shown in Fig 6.18, where compression cracks (fork cracks) are observed at the points of load application. Tensile cracks are formed at about 89% of the diameter.

Fig 6.19 – Siltstone sample 2-49-109.32 in tensile failure

Chapter 7: Presentation of Results & Discussions of Limestone

In this chapter, laboratory results of the various tests on limestone will be presented. Observations from AE non-destructive tests will also be included.

7.1 Physical Properties

The physical properties of limestone are determined in accordance with ISRM standard. The results of the specific gravity, absorption (as a percentage of dry weight) and surface moisture are shown in Table 7.1. (Refer to Appendix C6.1 for the detailed computation)

Table 7.1 – Physical properties of limestone

Specific gravity	Absorption (%)	Surface moisture (%)
2.914	0.246	0.0309

7.2 Petrographical Analysis of Thin Sections

The thin sections of limestone of *BH7A CR74* are analysed in X-Ray Fluorescence, XRF. The following subsections describe the limestone in a hand-specimen and under thin section.

Hand-specimen description

Colour: - Grey
Visible minerals: - Nil
Grain size: - Fine grained
Structures: - Massive, but with re-crystallised small cracks

Thin-section description

Mineral contents

The mineral contents (modal %) are semi-quantified based on point-counting of 150 points.

The compositions are as follows: - Quartz 10%, feldspar 28%, matrix material (materials is too small to be identified) 23%, carbonate 34%, opaque materials 4%.

Description of textures

The rock is a kind of carbonate breccia where the fragments compose of sand-sized quartz, feldspar and clay-sized carbonate. The matrix around the fragments composes of the same materials, but the portion of carbonate is much higher (Fig 7.1a). The carbonate minerals seem to be remobilized as infilling materials around the fragments. The brecciated textures are best recognized in plane polarized light (Fig 7.1b). The high content of barium (*Ba*) (Appendix C6.2) could be due to substitution of calcium (*Ca*) in the feldspar or in carbonate.

*Fig 7.1a – Carbonate breccia under XPL
Note the high content of carbonate in the matrix.
Ca = carbonate, Qz = quartz*

Fig 7.1b – Carbonate breccia under PPL

Additional descriptions given by the borehole log are as follows: -

- a) Light grey to grey and white;
- b) moderately weathered;
- c) *LIMESTONE* with mudstone pockets;
- d) strong to moderately strong,
- e) closely to very closely spaced normal to inclined fractures.

7.3 Compression Test

The results presented in the following section are based on samples of room moisture content tested under uniaxial and triaxial conditions.

7.3.1 Stress-Strain Curve

The deformation behaviour of the rock is presented in a form of a stress-strain curve and the mechanical properties of limestone will be discussed.

7.3.1.1 Axial and Lateral Strain Curves

The maximum axial, lateral and volumetric strain of limestone under various confining pressures are tabulated in Table 7.2. Appendix D7.1 summarises the test data for limestone samples.

Table 7.2 – Summary of maximum axial, lateral & volumetric strain of limestone

Confining pressure, σ_3 , MPa (No. of samples)	Maximum axial strain, $\epsilon_a(\text{max})$ %	Maximum lateral strain, $\epsilon_l(\text{max})$ %	Maximum volumetric strain $\epsilon_v(\text{max})$ %
0 (8)	0.129 (± 0.063)**	- 0.055 (± 0.030)	0.036 (± 0.033)
5 (2)*	0.163 (± 0.008)	- 0.096 (± 0.025)	0.025 (± 0.005)
10 (3)	0.166 (± 0.051)	- 0.098 (± 0.042)	0.026 (± 0.012)
20 (2)	0.186 (± 0.014)	- 0.115 (± 0.002)	0.012 (± 0.003)
30 (2)	0.244 (± 0.055)	- 0.145 (± 0.086)	0.023 (± 0.019)
40 (3)	0.336 (± 0.059)	- 0.345 (± 0.196)	0.012 (± 0.007)
Note: * $\sigma_3 = 5$ MPa, number of samples = 2; ** $\epsilon_a = 0.129\%$, standard deviation = 0.063%.			

The typical stress-strain curves at various confining pressures are illustrated in Fig 7.2 & 7.3. From the graphs, it is observed that the gradient of the curves for both axial and lateral strains increases with confining pressure. This shows that the confining pressure is limiting the amount of axial and lateral displacement. In addition, there is a distinct non-linear zone at the beginning of the curves, especially in the case of lateral strain. Comparing Fig 7.2 & 7.3 to Fig 6.2 & 6.3, both the stress-axial strain curves are similar, whereas the stress-lateral strain curves of siltstone show sign of flattening at higher strain.

Fig 7.2 – Typical stress-axial strain curves of limestone

Fig 7.3 – Typical stress-lateral strain curves of limestone

7.3.1.2 Young’s Modulus and Poisson’s Ratio

The Young’s modulus (E) and Poisson’s ratio (ν) of the rock samples are computed in the linear elastic deformation zone, at stress ranging from 12 MPa (unconfined) to 103 MPa (confining pressure of 40 MPa). A summary of results is shown in Table 7.3 and graphically illustrated in Fig 7.4.

Table 7.3 – Young’s modulus and Poisson’s ratio of limestone

Confining pressure, σ_3 (MPa)	Failure stress, σ_f (MPa)	Young’s modulus, E (GPa)	Poisson’s ratio, ν
0 (8)	112 ($\pm$ 45)	99.5 ($\pm$ 19.4)	0.371 ($\pm$ 0.079)
5 (2)*	152 ($\pm$ 26)**	105.9 ($\pm$ 6.2)	0.398 ($\pm$ 0.067)
10 (3)	168 ($\pm$ 63)	110.8 ($\pm$ 11.9)	0.417 ($\pm$ 0.098)
20 (2)	182 ($\pm$ 27)	118.8 ($\pm$ 13.6)	0.476 ($\pm$ 0.020)
30 (2)	195 ($\pm$ 19)	113.4 ($\pm$ 0.0)	0.448 ($\pm$ 0.047)
40 (3)	321 ($\pm$ 32)	117.5 ($\pm$ 4.8)	0.486 ($\pm$ 0.013)

Note: * $\sigma_3 = 5$ MPa, number of samples = 2;
** $\sigma_f = 152$ MPa, standard deviation = 26 MPa.
Value in parenthesis denotes standard deviation for σ_f , E and ν .

Fig 7.4 – Young’s modulus and Poisson’s ratio of limestone

The Young's modulus has an increasing trend with respect to the confining pressure till $\sigma_3 = 20$ MPa. Thereafter the Young's modulus remains constant with a slight decrease at $\sigma_3 = 30$ MPa.

Similarly for the Poisson's ratio, it exhibits an increasing trend with a slightly higher value at $\sigma_3 = 20$ MPa.

7.3.2 Stress Thresholds Determination

The following sections will present the various stress thresholds of limestone.

7.3.2.1 Crack Closure (σ_{cc})

Using axial strain technique on limestone sample 7A-72-166.51 (typical sample, $\sigma_3 = 10$ MPa), crack closure is found to be **15 MPa**. However, this point is not very obvious from the stress-strain curve as it shows very marginal non-linearity during the closure of cracks.

Using the axial stiffness technique, the crack closure of sample 7A-72-166.51 is determined to be **15 MPa** as shown in Fig 7.5 and this value agrees well with the value obtained in the above mentioned technique.

Fig 7.5 – Identification of stress threshold of crack closure using axial stiffness

7.3.2.2 Crack initiation (σ_{ci})

The crack initiation of a typical limestone sample (7A-72-166.51), using the lateral strain curve technique, is **63 MPa** as shown in Fig 7.6a.

Fig 7.6 – (a) Identification of stress thresholds using volumetric & lateral strains

Using the calculated crack volumetric strain, the corresponding crack initiation at the point where dilation begins after crack volume remain unchanged is approximately **74 MPa** (Fig 7.6b). The latter method gives a higher stress threshold and a better representation of the stress threshold as it is determined using the crack volumetric strain, which is calculated using Young’s modulus and Poisson’s ratio, compared to the former method which considers lateral strain behaviour only.

Fig 7.7 –(b) Identification of stress thresholds using crack volumetric strain

7.3.2.3 Crack Coalescence (σ_{cs})

From Fig 7.7, the crack coalescence of sample 7A-72-166.51 is found to be **89 MPa** which reflects the point whereby the volumetric stiffness curve starts to show large irregularities.

7.3.2.4 Crack Damage (σ_{cd})

The crack damage of sample 7A-72-166.51 is approximately **85 MPa**, determined from the point of reversal of the volumetric strain curve (Fig 7.6a). However, this phenomenon is not observed in few of the limestone samples.

Using volumetric stiffness curve technique (Eberhardt et al., 1998), it is very distinct, as shown in Fig 7.7, that the crack damage of sample 7A-72-166.51 is **100 MPa** which is higher than that determined previously using volumetric strain curve method.

7.3.2.5 Failure Strength

The failure strength is taken as the peak stress the sample has been subjected to. Limestone sample 7A-72-166.51 has a failure strength of **137 MPa**. (Fig 7.6a).

7.3.3 Stress Threshold Results

A summary of the various stress thresholds is shown in Table 7.4 and Fig 7.8.

Table 7.4 – Stress thresholds of limestone

Confining Pressure, σ_3 (MPa)		Crack closure, σ_{cc} (MPa)	Crack initiation, σ_{ci} (MPa)	Crack coalescence, σ_{cs} (MPa)	Crack damage, σ_{cd} (MPa)	Failure, σ_{if} (MPa)
0 (8)	σ	12 (± 4)	41 (± 19)	73 (± 42)	79 (± 41)	112 (± 45)
	σ / σ_f	0.11	0.37	0.65	0.70	1.00
5* (2)	σ	17 (± 2)**	85 (± 29)	104 (± 21)	110 (± 18)	152 (± 26)
	σ / σ_f	0.11	0.56	0.68	0.72	1.00
10 (3)	σ	18 (± 7)	88 (± 54)	109 (± 42)	117 (± 37)	168 (± 63)
	σ / σ_f	0.11	0.53	0.65	0.69	1.00
20 (2)	σ	22 (± 2)	89 (± 23)	105 (± 4)	111 (± 3)	182 (± 27)
	σ / σ_f	0.12	0.49	0.58	0.61	1.00
30 (2)	σ	23 (± 5)	87 (± 19)	119 (± 19)	129 (± 28)	195 (± 19)
	σ / σ_f	0.12	0.44	0.61	0.66	1.00
40 (3)	σ	35 (± 4)	103 (± 25)	134 (± 47)	146 (± 58)	321 (± 32)
	σ / σ_f	0.11	0.32	0.42	0.45	1.00
Note: * $\sigma_3 = 5$ MPa, number of samples = 2; ** $\sigma_{cc} = 17$ MPa, standard deviation = 2 MPa. Value in parenthesis denotes standard deviation for stress thresholds.						

The average stress threshold of crack closure (σ_{cc}), crack initiation (σ_{ci}), crack coalescence (σ_{cs}) and crack damage (σ_{cd}) is 0.11, 0.45, 0.60 and 0.64 of the failure strength (σ_f), respectively.

Fig 7.9 – Stress thresholds of limestone

7.3.3.1 Failure Criterion

The experimental results are fitted using the Hoek-Brown failure criterion (Hoek et al., 1995).

Fig 7.10 – Hoek-Brown failure criterion of limestone

Figure 7.9 shows a plot of the Hoek-Brown failure criterion established from the experimental results. An m value of **17** (correlation = 0.70) with a uniaxial compressive strength (σ_c) of **112 MPa** is able to fit the Hoek-Brown failure envelope to the experimental results. The m value is obtained by the best fit method and verified by the *ROCKDATA Version 2.21* software. At $\sigma_3 = 20$ and 30 MPa, Hoek-Brown envelope tends to over-estimate the results in correlation to that of the experimental ones.

The *ROCKDATA Version 2.21* software developed by Rock Engineering Group, University of Toronto was used to obtain the parameters for the Mohr-Coulomb failure criterion. The friction angle (ϕ) determined is **43.70°**

and the cohesion (c) is **24.2 MPa**, i.e. $\tau = 24.2 + \sigma_n \tan 43.7^\circ$. The results display of the software is shown in Figure 7.10.

Fig 7.11 – Mohr-Coulomb failure criterion of limestone

7.3.4 Acoustic Emission Tests

AE tests which were carried out concurrently with the compression tests are presented in the following section.

7.3.4.1 Acoustic Emission Counts

The detected signals of crack initiations and propagations are filtered into high and low frequency channels. The amplitudes of high frequency channel are detected and those surpassing the background threshold set at 500 mV are registered as ring down counts. The cumulative acoustic emission ring down count is plotted against stress as shown in Fig 7.11.

Fig 7.12 – Graph of cumulative emission ring down counts versus stress

For limestone sample 7A-72-166.51 (typical sample, $\sigma_3 = 10$ MPa), AE counts was first detected at **15 MPa** which coincides with the stress threshold of crack closure (σ_{cc}) that was determined previously, using the axial strain method suggested by Bieniawski (1967b).

From σ_{cc} onwards, few AE counts are registered. More AE counts are then detected at **63 MPa** which corresponds to the stress threshold of crack initiation (σ_{ci}). The AE counts increase linearly from this point before it departs into a steeper slope at around **89 MPa**. This point is identified as the stress threshold of crack coalescence (σ_{cs}).

After σ_{cs} , the AE counts increase linearly upwards at a constant gradient until it reaches a point whereby it deviates from linearity. This point is the stress threshold of crack damage (σ_{cd}) which is identified as **100 MPa**. It is observed that the various stress thresholds agree relatively well with that

determined earlier using the stress-strain techniques ($\sigma_{cc} = 15$ MPa, $\sigma_{ci} = 74$ MPa, $\sigma_{cs} = 89$ MPa, $\sigma_{cd} = 100$ MPa).

7.3.5 Failure Mode

Similar to siltstone, there are basically two main types of failure mode observed in uniaxial and triaxial compression tests, namely slabbing and shearing.

A typical sample of a *slabbing* failure mode is shown in Fig 7.12. It is observed that uniaxial tests are more prone to this type of failure.

Fig 7.13 – Slabbing failure of limestone sample 7A-72-167.46 ($\sigma_3 = 0$ MPa)

In the case of triaxial tests, the *shearing* failure mode is more dominant. Fig 7.13 shows a typical sample of a shearing failure which fails at a single oblique plane.

Fig 7.14 – Shearing failure of limestone sample 7A-72-168.08 ($\sigma_3 = 40$ MPa)

7.4 Tensile Test

The average tensile strength of limestone is **13 MPa** (± 3 MPa) based one 11 samples tested in Brazilian Test as shown in Appendix D7.2. A typical stress-strain curve of a Brazilian Test on limestone is shown in Fig 7.14.

Fig 7.15 – Brazilian Test on limestone

The stress corresponding to the point of departure from non-linearity to linearity is the stress threshold of crack initiation (σ_{ci}) (Fig 7.15). The initial non-linear part of the curve represents the crack closure stage. For sample 7A-71-164.24, σ_{ci} is **4.2 MPa** ($0.37 \sigma_f$) and the rest of the samples have an average value of **0.41 σ_f** .

Acoustic emission is monitored during the Brazilian test to study the cracking behaviour of rocks under tensile stress. A plot of summation of counts versus tensile stress is shown in Fig 7.15.

Fig 7.16 – Acoustic emission test on limestone under tensile stress

From Fig 7.15, the first AE count is registered at a tensile stress of **4.2 MPa**, which agrees with the stress threshold of crack initiation (σ_{ci}) as determined above (Fig 7.14). Hence, it can be concluded that, in this case, when σ_{ci} is reached, AE will start to occur. Despite the few AE counts in the AE versus stress plot, the stress threshold of crack damage (σ_{cd}) is still able to be determined at a value of **9.6 MPa** ($0.84 \sigma_f$).

The above illustrated result is one of the few limestone samples to have this phenomenon. As for the remaining test results, a few of them show that there are very little or no AE counts recorded throughout the Brazilian test.

An example of a failed sample is shown in Fig 7.16, where compression cracks are observed at the points of load application. In addition, the sample failed by splitting through the centre into approximately symmetrical halves.

Fig 7.17 – Limestone sample 7A-71-164.24 in tensile failure

Chapter 8: Presentation of Results & Discussions of Sandstone

In this chapter, laboratory results of the various tests on sandstone will be presented. Observations from AE non-destructive tests and ultrasonic tests will also be included.

8.1 Physical Properties

The physical properties of sandstone are determined in accordance with ISRM standard. The results of the specific gravity, absorption (as a percentage of dry weight) and surface moisture are shown in Table 8.1. (Refer to Appendix C6.1 for the detailed computation)

Table 8.1 – Physical properties of sandstone

Specific gravity	Absorption (%)	Surface moisture (%)
2.734	0.334	0.0453

8.2 Petrographical Analysis of Thin Sections

The thin sections of sandstone of BH3 CR42 are analysed in X-Ray Fluorescence, XRF. The following subsections describe the sandstone in a hand-specimen and under thin section.

Hand-specimen description

Colour: - Dark greyish with lamina
Visible minerals: - Nil
Grain size: - Fine
Structures: - Massive

Thin-section description

Mineral contents

The mineral contents (modal %) are semi-quantified based on point-counting of 150 points.

The compositions are as follows: - Quartz 10%, feldspar 30%, matrix (materials is too small to be identified) 43%, carbonate 5%, muscovite 5% opaque materials 8%.

Description of textures

The rock shows a lamina defined by change in grain size and colour (Fig 8.1). The dark layers (matrix) compose of clay-sized materials, where the lighter bands consist of sand sized quartz and feldspar and small amounts of carbonate and muscovite. The quartz and feldspar grains are angular in shape. The opaque material is oriented parallel to the lamina. The rounded light grains in the matrix are calcite and silica. The shape indicates that they may represent remains of some micro-organism. Some small micro cracks are filled with carbonate.

Fig 8.1 – Alternation between dark and light layers of sandstone under XPL

Additional descriptions given by the borehole log are as follows: -

- a) Grey to dark grey;
- b) speckled weathered fine to medium;
- c) *SANDSTONE*;
- d) partly inter-bedded with mudstone,

- e) very strong to strong;
- f) normal to inclined fractures (SW); and
- g) dip angle = 11 degree.

8.3 **Compression Test**

The results presented in the following section are based on samples of room moisture content tested under uniaxial and triaxial conditions.

8.3.1 **Stress-Strain Curve**

The deformation behaviour of the rock is presented in a form of a stress-strain curve and the mechanical properties of sandstone will be discussed.

8.3.1.1 **Axial and Lateral Strain Curves**

The maximum axial, lateral and volumetric strain of sandstone under various confining pressures are tabulated in Table 8.2. Appendix E8.1 summarises the test data for sandstone samples.

Table 8.2 – Summary of maximum axial, lateral & volumetric strain of sandstone

Confining pressure, σ_3 , MPa (No. of samples)	Maximum axial strain, $\epsilon_a(\text{max})$ %	Maximum lateral strain, $\epsilon_l(\text{max})$ %	Maximum volumetric strain $\epsilon_v(\text{max})$ %
0 (9)	0.161 (± 0.074)**	- 0.108 (± 0.063)	0.045 (± 0.042)
5 (6)*	0.209 (± 0.054)	- 0.193 (± 0.037)	0.047 (± 0.020)
10 (8)	0.269 (± 0.043)	- 0.255 (± 0.156)	0.053 (± 0.024)
20 (5)	0.326 (± 0.050)	- 0.269 (± 0.106)	0.060 (± 0.066)
30 (6)	0.361 (± 0.061)	- 0.277 (± 0.183)	0.109 (± 0.144)
40 (5)	0.375 (± 0.028)	- 0.293 (± 0.126)	0.132 (± 0.099)
Note: * $\sigma_3 = 5$ MPa, number of samples = 6; ** $\epsilon_a = 0.161\%$, standard deviation = 0.074%.			

The typical stress-strain curves at various confining pressures are illustrated in Fig 8.2 & 8.3. From the graphs, it is observed that the gradient of the curves for both axial and lateral strains increases with confining pressure. In the case of the axial strain, an increase in gradient implies an increase in Young’s modulus as well, which will be presented in the next section.

In addition, the maximum axial, lateral as well as volumetric strain increase with confining pressure. (See Table 8.2) This shows that the confining pressure is limiting the amount of axial and lateral displacement.

Fig 8.2 – Typical stress-axial strain curves of sandstone

Fig 8.3 – Typical stress-lateral strain curves of sandstone

8.3.1.2 Young’s Modulus and Poisson’s Ratio

The Young’s modulus (E) and Poisson’s ratio (v) of the rock samples are computed in the linear elastic deformation zone, at stress ranging from 17 MPa (unconfined) to 135 MPa (confining pressure of 40 MPa). A summary of results is shown in Table 8.3 and graphically illustrated in Fig 8.4.

Table 8.3 – Young’s modulus and Poisson’s ratio of sandstone

Confining pressure, σ_3 (MPa)	Failure stress, σ_f (MPa)	Young’s modulus, E (GPa)	Poisson’s ratio, v
0 (9)	129 (± 62)	83.2 (± 10.9)	0.350 (± 0.032)
5 (6)*	139 (± 68)**	86.6 (± 19.7)	0.351 (± 0.025)
10 (8)	183 (± 41)	87.1 (± 10.6)	0.396 (± 0.006)
20 (5)	268 (± 25)	87.9 (± 8.7)	0.422 (± 0.035)
30 (6)	325 (± 50)	92.5 (± 4.8)	0.434 (± 0.008)
40 (5)	372 (± 47)	102.6 (± 10.6)	0.456 (± 0.038)

Note: * σ_3 = 5 MPa, number of samples = 6;
 ** σ_f = 139 MPa, standard deviation = 68 MPa.
 Value in parenthesis denotes standard deviation for σ_f , E and v.

Fig 8.4 – Young's modulus and Poisson's ratio of sandstone

The Young's modulus has an increasing trend with respect to the confining pressure. Similarly, the Poisson's ratio exhibits an increasing trend.

8.3.2 Stress Thresholds Determination

The following sections will present the various stress thresholds of sandstone.

8.3.2.1 Crack Closure (σ_{cc})

Using axial strain technique on sandstone sample 3-43-4 (typical sample, $\sigma_3 = 10$ MPa), crack closure is found to be **23 MPa**. However, this point is not very obvious from the stress-strain curve as it shows very marginal non-linearity during the closure of cracks.

Using the axial stiffness technique, the crack closure of sample 3-43-4 is determined to be **23 MPa** as shown in Fig 8.5 and this value agrees well with the value obtained in the above-mentioned technique.

Fig 8.5 – Identification of stress threshold of crack closure using axial stiffness

8.3.2.2 Crack initiation (σ_{ci})

The crack initiation of a typical sandstone sample (3-43-4), using the lateral strain curve technique, is **69 MPa** as shown in Fig 8.6a. Using the calculated crack volumetric strain, the corresponding crack initiation at the point where dilation begins after crack volume remain unchanged is approximately **57 MPa** (Fig 8.6b). Both values obtained through the 2 techniques agree relatively well with each other, with the latter method giving a 21% lower value.

**Fig 8.6 – (a) Identification of stress thresholds using volumetric & lateral strains;
(b) Identification of stress thresholds using crack volumetric strain**

8.3.2.3 Crack Coalescence (σ_{cs})

From Fig 8.7, the crack coalescence of sample 3-43-4 is found to be **152 MPa** which reflects the point whereby the volumetric stiffness curve starts to show large irregularities.

Fig 8.7 – Identification of stress thresholds using volumetric stiffness

8.3.2.4 Crack Damage (σ_{cd})

The crack damage of sample 3-43-4 is **90 MPa**, determined from the point of reversal of the volumetric strain curve (Fig 8.6a). However, this phenomenon is not observed in few of the sandstone samples.

Using volumetric stiffness curve technique (Eberhardt et al., 1998), it is very distinct, as shown in Fig 8.7, that the crack damage of sample 3-43-4 is **174 MPa** which is higher than that determined previously using volumetric strain curve method.

8.3.2.5 Failure Strength

The failure strength is taken as the peak stress the sample has been subjected to. Sandstone sample 3-43-4 has a failure strength of **221 MPa**. (Fig 8.6a).

8.3.3 Stress Threshold Results

A summary of the various stress thresholds is shown in Table 8.4 and Fig 8.8.

Table 8.4 – Stress thresholds of sandstone

Confining Pressure, σ_3 (MPa)		Crack closure, σ_{cc} (MPa)	Crack initiation, σ_{ci} (MPa)	Crack coalescence, σ_{cs} (MPa)	Crack damage, σ_{cd} (MPa)	Failure, σ_{lf} (MPa)
0 (9)	σ	17 ($\pm$ 5)	56 ($\pm$ 27)	72 ($\pm$ 35)	85 ($\pm$ 46)	129 ($\pm$ 62)
	σ / σ_f	0.13	0.43	0.56	0.66	1.00
5* (6)	σ	21 ($\pm$ 10)**	59 ($\pm$ 29)	74 ($\pm$ 38)	94 ($\pm$ 52)	136 ($\pm$ 68)
	σ / σ_f	0.15	0.43	0.54	0.63	1.00
10 (8)	σ	28 ($\pm$ 8)	73 ($\pm$ 17)	95 ($\pm$ 21)	130 ($\pm$ 36)	173 ($\pm$ 41)
	σ / σ_f	0.16	0.42	0.55	0.70	1.00
20 (5)	σ	38 ($\pm$ 12)	103 ($\pm$ 13)	139 ($\pm$ 14)	212 ($\pm$ 22)	248 ($\pm$ 25)
	σ / σ_f	0.15	0.41	0.56	0.85	1.00
30 (6)	σ	50 ($\pm$ 12)	118 ($\pm$ 17)	171 ($\pm$ 29)	229 ($\pm$ 62)	295 ($\pm$ 50)
	σ / σ_f	0.17	0.40	0.58	0.77	1.00
40 (5)	σ	68 ($\pm$ 18)	135 ($\pm$ 31)	199 ($\pm$ 28)	268 ($\pm$ 37)	372 ($\pm$ 47)
	σ / σ_f	0.21	0.41	0.60	0.81	1.00
Note: * σ_3 = 5 MPa, number of samples = 6; ** σ_{cc} = 21 MPa, standard deviation = 10 MPa. Value in parenthesis denotes standard deviation for stress thresholds.						

The average stress threshold of crack closure (σ_{cc}), crack initiation (σ_{ci}), crack coalescence (σ_{cs}) and crack damage (σ_{cd}) is 0.16, 0.42, 0.56 and 0.74 of the failure strength (σ_f), respectively.

Fig 8.8 – Stress thresholds of sandstone

8.3.3.1 Failure Criterion

The experimental results are fitted using the Hoek-Brown failure criterion (Hoek et al., 1995).

Figure 8.9 shows a plot of the Hoek-Brown failure criterion established from the experimental results. An m value of **14.0** (correlation = 0.67) with a uniaxial compressive strength (σ_c) of **129 MPa** is able to fit the Hoek-Brown failure envelope to the experimental results. The m value is obtained by the best fit method and verified by the *ROCKDATA Version 2.21* software. Some experimental values are lower than the average as the samples maybe subjected to higher weathering rate and thus weaker in nature.

The *ROCKDATA Version 2.21* software developed by Rock Engineering Group, University of Toronto was used to obtain the parameters for the Mohr-Coulomb failure criterion. The friction angle (ϕ) determined is **43.8°**

and the cohesion (c) is **26.1 MPa**, i.e. $\tau = 26.1 + \sigma_n \tan 43.8^\circ$. The results display of the software is shown in Figure 8.10.

Fig 8.10 – Mohr-Coulomb failure criterion of sandstone

8.3.4 Acoustic Emission Tests

AE tests which were carried out concurrently with the compression tests are presented in the following section.

8.3.4.1 Acoustic Emission Counts

Sample 3-42-3 is used as a typical example to illustrate the behaviour of AE counts under uniaxial loading ($\sigma_3 = 0 \text{ MPa}$). The cumulative acoustic emission ring down count is plotted against stress as shown in Fig 8.11.

Fig 8.11 – Graph of cumulative emission ring down counts versus stress

The initial AE counts detected represent the crack closure stage, and these AE counts ceased at a stress of 22 MPa (σ_{cc}). Subsequently, no AE count is detected until a stress of 48 MPa (σ_{ci}). This region bounded by σ_{cc} and σ_{ci} is known to be the elastic range. From σ_{ci} onwards, more AE counts are detected, as the propagation of cracks occurs till a stress of 57 MPa (σ_{cd}), where an increase in slope signifies a more rapid propagation of cracks, all the way till failure. In comparison of the stress thresholds determined using the stress-strain technique ($\sigma_{cc} = 22$ MPa; $\sigma_{ci} = 48$ MPa and $\sigma_{cd} = 57$ MPa) with that of the AE technique, the values agree very well.

The above illustrated example is only one of the few tests with these characteristics. Some of the tests have no AE counts detected throughout, while some have little AE counts detected towards the end of the test. In the latter case, it is observed that when the stress threshold of crack damage (σ_{cd}) is reached, AE counts can be detected. Thus, the AE method is only

effective in determining σ_{cd} of sandstone, as for the rest of the stress thresholds; it should be evaluated using other methods.

8.3.5 Ultrasonic Tests

In addition to AE tests, the ultrasonic tests are conducted to investigate the behaviour of P&S wave velocities during compression tests.

8.3.5.1 Compression (P) Wave Velocity

Sample 3-43-4, used to illustrate the stress-strain technique in the above discussions, is tested with P-wave velocity. Fig 8.12 shows the variation of P-wave velocities with deviator stress.

Fig 8.12 – P-wave velocity versus stress (3-43-4)

From the graph shown above, σ_{cc} , σ_{ci} and σ_{cd} are found to be approximately 23 MPa, 71 MPa and 114 MPa respectively. The stress threshold of crack coalescence (σ_{cs}) could not be determined from the ultrasonic tests. To recap, the various stress thresholds determined using the stress-strain techniques are as follows: - $\sigma_{cc} = 23$ MPa; $\sigma_{ci} = 69$ MPa and $\sigma_{cd} = 110$ MPa. Therefore, it can be seen that the stress thresholds agree well for both

methods. Further tests conducted at uniaxial and triaxial test at various confining pressures show similar findings.

8.3.5.2 Shear (S) Wave Velocity

Sample 3-45-4 is used as a typical example to illustrate the behaviour of S wave under triaxial loading ($\sigma_3 = 5$ MPa).

Fig 8.13 – S-wave velocity versus stress (3-45-4)

From the graph shown above, σ_{cc} , σ_{ci} and σ_{cd} are found to be approximately 28 MPa, 73 MPa and 155 MPa respectively. The stress threshold of crack coalescence (σ_{cs}) could not be determined from the ultrasonic tests. Comparing these values with that of the stress-strain techniques ($\sigma_{cc} = 32$ MPa; $\sigma_{ci} = 66$ MPa and $\sigma_{cd} = 147$ MPa), there is a variation of less than 10%. Nevertheless, the S-wave test correlates well to the stress-strain methods to determine the various stress thresholds except for σ_{cs} .

8.3.6 Failure Mode

Similar to siltstone and limestone, there are basically two main types of failure mode observed in uniaxial and triaxial compression tests on sandstone, namely slabbing and shearing.

A typical sample of a *slabbing* failure mode is shown in Fig 8.14. It is observed that uniaxial tests are more prone to this type of failure.

Fig 8.14 – Slabbing failure of sandstone sample 3-42-3 ($\sigma_3 = 0$ MPa)

In the case of triaxial tests, the *shearing* failure mode is more dominant. Fig 8.15 shows a typical sample of a shearing failure which fails at a single oblique plane.

Fig 8.15 – Shearing failure of sandstone sample 3-43-4 ($\sigma_3 = 10$ MPa)

8.4 Tensile Test

The average tensile strength of sandstone is **16 MPa** (± 4 MPa) based one 20 samples tested in Brazilian Test as shown in Appendix E8.2. A typical stress-strain curve of a Brazilian Test on sandstone is shown in Fig 8.16.

Fig 8.16 – Brazilian Test on sandstone

The stress corresponding to the point of departure from non-linearity to linearity is the stress threshold of crack initiation (σ_{ci}) (Fig 8.16). The initial non-linear part of the curve represents the crack closure stage. For sample 3-31-1, σ_{ci} is **3.54 MPa** ($0.40 \sigma_f$) and the rest of the samples have an average value of **0.50 σ_f** .

Acoustic emission is monitored during the Brazilian test to study the cracking behaviour of rocks under tensile stress. A plot of summation of counts versus tensile stress is shown in Fig 8.17.

Fig 8.17 – Acoustic emission test on sandstone under tensile stress

From Fig 8.17, the first AE count is registered at a tensile stress of **4.6 MPa**, which is higher than the stress threshold of crack initiation ($\sigma_{ci} = 3.64$ MPa) as determined above (Fig 8.16). However, there is no distinct feature in the AE versus stress plot to determine the stress threshold of crack damage (σ_{cd}). The 26% difference between the AE and tensile test method can be due to the sensitivity/ filter settings of the AE sensors to detect the first count. Higher sensitivity setting is not advisable as extraneous noise will be detected as well and clean data acquisition is not possible.

The above illustrated result is one of the few sandstone samples to have this phenomenon. As for the remaining test results, a few of them show that there are very little or no AE counts recorded throughout the Brazilian test.

An example of a failed sample is shown in Fig 8.18, where compression cracks are observed at the points of load application. Tensile cracks are formed in the direction of load application at about $0.90D$.

Fig 8.18 – Sandstone sample 3-31-1 in tensile failure

Chapter 9: Presentation of Results & Discussions of Mudstone

In this chapter, laboratory results of the various tests on mudstone will be presented. Observations from AE non-destructive tests will also be included.

9.1 Physical Properties

The physical properties of mudstone are determined in accordance with ISRM standard. The results of the specific gravity, absorption (as a percentage of dry weight) and surface moisture are shown in Table 9.1. (Refer to Appendix C6.1 for the detailed computation)

Table 9.1 – Physical properties of mudstone

Specific gravity	Absorption (%)	Surface moisture (%)
2.751	0.189	0.0137

9.2 Petrographical Analysis of Thin Sections

The thin sections of mudstone of *BH7 CR59* are analysed in X-Ray Fluorescence, XRF. The following subsections describe the mudstone in a hand-specimen and under thin section.

Hand-specimen description

- Colour: - Dark greyish
- Visible minerals: - Nil
- Grain size: - Extremely fine
- Structures: - Massive

Thin-section description

Mineral contents

It is impossible to give a semi quantification of mineral content (modal %) based on point-counting of minerals as the grains are too small to identify.

However, visible minerals are a sheet silicate possibly muscovite, carbonates, and opaque materials.

Description of textures

The thin section shows a kind of deformation structure probably as a result of slumping. The structure is defined by light materials in a dark clay-rich matrix (Fig 9.1a). The light material composes of mostly carbonate. There is also a relatively high proportion of dark minerals which could be related to the iron content. As for the other rock samples, the content of *Si* and *Al* (Appendix C6.2) are so high that the clay-fraction mostly consist of silicates. Figure 9.1b shows a mudstone at a higher magnification in plain polarised light (PPL). The small rounded white grains could be remains of micro-organism, but are now replaced by calcite.

Fig 9.1a – Deformation structure of mudstone under XPL

Fig 9.1b – Mudstone at a higher magnification under PPL

Additional descriptions given by the borehole log are as follows: -

- a) Grey to dark grey;
- b) slightly weathered;
- c) *MUDSTONE* inter-bedded with fine sandstone (meta-sedimentary rock);
- d) very strong,
- e) medium to closely spaced normal to inclined fractures (SW); and
- f) dip angle = 26 degree.

9.3 Compression Test

The results presented in the following section are based on samples of room moisture content tested under uniaxial and triaxial conditions.

9.3.1 Stress-Strain Curve

The deformation behaviour of the rock is presented in a form of a stress-strain curve and the mechanical properties of mudstone will be discussed.

9.3.1.1 Axial and Lateral Strain Curves

The maximum axial, lateral and volumetric strain of mudstone under various confining pressures are tabulated in Table 9.2. Appendix F9.1 summarises the test data for mudstone samples.

Table 9.2 – Summary of maximum axial, lateral & volumetric strain of mudstone

Confining pressure, σ_3 , MPa (No. of samples)	Maximum axial strain, $\epsilon_a(\text{max})$ %	Maximum lateral strain, $\epsilon_l(\text{max})$ %	Maximum volumetric strain $\epsilon_v(\text{max})$ %
0 (3)	0.155 (± 0.098)**	- 0.056 (± 0.063)	0.010 (± 0.006)
5 (2)*	0.168 (± 0.078)	- 0.082 (± 0.071)	0.095 (± 0.063)
10 (2)	0.240 (± 0.051)	- 0.106 (± 0.025)	0.097 (± 0.014)
20 (2)	0.301 (± 0.075)	- 0.173 (± 0.005)	0.101 (± 0.074)
30 (4)	0.309 (± 0.072)	- 0.193 (± 0.088)	0.107 (± 0.083)
40 (3)	0.390 (± 0.035)	- 0.202 (± 0.083)	0.112 (± 0.056)
Note: * $\sigma_3 = 5$ MPa, number of samples = 2; ** $\epsilon_a = 0.155\%$, standard deviation = 0.098%.			

The typical stress-strain curves at various confining pressures are illustrated in Fig 9.2 & 9.3. From the graphs, it is observed that the gradient of the curves for both axial and lateral strains increases with confining pressure. In the case of the axial strain, an increase in gradient implies an increase in Young’s modulus as well, which will be presented in the next section.

In addition, the maximum axial, lateral as well as volumetric strain increase with confining pressure. (See Table 9.2) This shows that the samples are able to deform more at higher confining pressure prior to failure.

Fig 9.2 – Typical stress-axial strain curves of mudstone

Fig 9.3 – Typical stress-lateral strain curves of mudstone

9.3.1.2 Young’s Modulus and Poisson’s Ratio

The Young’s modulus (E) and Poisson’s ratio (ν) of the rock samples are computed in the linear elastic deformation zone, at stress ranging from 11 MPa (unconfined) to 101 MPa (confining pressure of 40 MPa). A summary of results is shown in Table 9.3 and graphically illustrated in Fig 9.4.

Table 9.3 – Young’s modulus and Poisson’s ratio of mudstone

Confining pressure, σ_3 (MPa)	Failure stress, σ_f (MPa)	Young’s modulus, E (GPa)	Poisson’s ratio, ν
0 (3)	91 (± 28)	87.8 (± 1.5)	0.162 (± 0.006)
5 (2)*	143 (± 52)**	91.8 (± 2.2)	0.164 (± 0.044)
10 (2)	200 (± 7)	93.2 (± 1.6)	0.248 (± 0.040)
20 (2)	301 (± 6)	95.8 (± 3.6)	0.312 (± 0.016)
30 (4)	312 (± 25)	97.1 (± 3.0)	0.454 (± 0.042)
40 (3)	334 (± 36)	103.4 (± 7.0)	0.492 (± 0.011)

Note: * $\sigma_3 = 5$ MPa, number of samples = 2;
** $\sigma_f = 139$ MPa, standard deviation = 68 MPa.
Value in parenthesis denotes standard deviation for σ_f , E and ν .

Fig 9.4 – Young’s modulus and Poisson’s ratio of mudstone

The Young’s modulus has an increasing trend with respect to the confining pressure. Similarly, the Poisson’s ratio exhibits an increasing trend, to a point whereby the value approaches 0.5 at 40 MPa confining pressure. This phenomenon will be discussed in Chapter 10.

9.3.2 Stress Thresholds Determination

The following sections will present the various stress thresholds of mudstone.

9.3.2.1 Crack Closure (σ_{cc})

Using axial strain technique on mudstone sample 7-58-4 (typical sample, $\sigma_3 = 30$ MPa), crack closure is found to be **40 MPa**. However, this point is not very obvious from the stress-strain curve as it shows very marginal non-linearity during the closure of cracks.

Using the axial stiffness technique, the crack closure of sample 7-58-4 is determined to be **40 MPa** as shown in Fig 9.5 and this value agrees well with the value obtained in the above mentioned technique.

Fig 9.5 – Identification of stress threshold of crack closure using axial stiffness

9.3.2.2 Crack initiation (σ_{ci})

The crack initiation of a typical mudstone sample (7-58-4), using the lateral strain curve technique, is **104 MPa** as shown in Fig 9.6a.

Fig 9.6 – (a) Identification of stress thresholds using volumetric & lateral strains

Using the calculated crack volumetric strain, the corresponding crack initiation at the point where dilation begins after crack volume remain unchanged is approximately **51 MPa** (Fig 9.6b). The latter method gives a higher stress threshold and a better representation of the stress threshold as it is determined using the crack volumetric strain, which is calculated using Young’s modulus and Poisson’s ratio, compared to the former method which considers lateral strain behaviour only.

Fig 9.7 – (b) Identification of stress thresholds using crack volumetric strain

9.3.2.3 Crack Coalescence (σ_{cs})

From Fig 9.7, the crack coalescence of sample 7-58-4 is found to be **284 MPa** which reflects the point whereby the volumetric stiffness curve starts to show large irregularities.

Fig 9.8 – Identification of stress thresholds using volumetric stiffness

9.3.2.4 Crack Damage (σ_{cd})

The crack damage of sample 7-58-4 is **140 MPa**, determined from the point of reversal of the volumetric strain curve (Fig 9.6a). However, this phenomenon is not observed in few of the mudstone samples.

Using volumetric stiffness curve technique (Eberhardt et al, 1998), it is very distinct, as shown in Fig 9.7, that the crack damage of sample 7-58-4 is **300 MPa** which is higher than that determined previously using volumetric strain curve method.

9.3.2.5 Failure Strength

The failure strength is taken as the peak stress the sample has been subjected to. Mudstone sample 7-58-4 has a failure strength of **312 MPa**. (Fig 9.6a).

9.3.3 Stress Threshold Results

A summary of the various stress thresholds is shown in Table 9.4 and Fig 9.8.

Table 9.4 – Stress thresholds of mudstone

Confining Pressure, σ_3 (MPa)		Crack closure, σ_{cc} (MPa)	Crack initiation, σ_{ci} (MPa)	Crack coalescence, σ_{cs} (MPa)	Crack damage, σ_{cd} (MPa)	Failure, σ_{lf} (MPa)
0 (3)	σ	11 ($\pm$ 1)	32 ($\pm$ 5)	64 ($\pm$ 20)	86 ($\pm$ 32)	91 ($\pm$ 28)
	σ / σ_f	0.12	0.35	0.70	0.94	1.00
5* (2)	σ	12 ($\pm$ 0)**	62 ($\pm$ 8)	107 ($\pm$ 47)	137 ($\pm$ 51)	143 ($\pm$ 52)
	σ / σ_f	0.09	0.45	0.77	0.99	1.00
10 (2)	σ	20 ($\pm$ 1)	70 ($\pm$ 5)	134 ($\pm$ 4)	156 ($\pm$ 7)	200 ($\pm$ 7)
	σ / σ_f	0.10	0.37	0.70	0.82	1.00
20 (2)	σ	27 ($\pm$ 2)	76 ($\pm$ 1)	198 ($\pm$ 3)	278 ($\pm$ 8)	301 ($\pm$ 6)
	σ / σ_f	0.10	0.27	0.70	0.99	1.00
30 (4)	σ	31 ($\pm$ 9)	82 ($\pm$ 19)	207 ($\pm$ 84)	246 ($\pm$ 76)	312 ($\pm$ 25)
	σ / σ_f	0.11	0.29	0.72	0.86	1.00
40 (3)	σ	35 ($\pm$ 7)	101 ($\pm$ 9)	210 ($\pm$ 9)	265 ($\pm$ 11)	334 ($\pm$ 36)
	σ / σ_f	0.12	0.34	0.71	0.90	1.00
Note: * σ_3 = 5 MPa, number of samples = 2; ** σ_{cc} = 12 MPa, standard deviation = 0 MPa. Value in parenthesis denotes standard deviation for stress thresholds.						

The average stress threshold of crack closure (σ_{cc}), crack initiation (σ_{ci}), crack coalescence (σ_{cs}) and crack damage (σ_{cd}) is 0.11, 0.34, 0.72 and 0.94 of the failure strength (σ_f), respectively.

Fig 9.9 – Stress thresholds of mudstone

9.3.3.1 Failure Criterion

The experimental results are fitted using the Hoek-Brown failure criterion (Hoek et al, 1995).

Fig 9.10 – Hoek-Brown failure criterion of mudstone

Figure 9.9 shows a plot of the Hoek-Brown failure criterion established from the experimental results. An m value of **12.0** (correlation = 0.72) with a uniaxial compressive strength (σ_c) of **91 MPa** is able to fit the Hoek-Brown failure envelope to the experimental results. The m value is obtained by the best fit method and verified by the *ROCKDATA Version 2.21* software. Some experimental values are lower than the average as the samples maybe subjected to higher weathering rate and thus weaker in nature.

The *ROCKDATA Version 2.21* software developed by Rock Engineering Group, University of Toronto was used to obtain the parameters for the Mohr-Coulomb failure criterion. The friction angle (ϕ) determined is **42.4°**

and the cohesion (c) is **26.3 MPa**, i.e. $\tau = 26.3 + \sigma_n \tan 42.4^\circ$. The results display of the software is shown in Figure 9.10.

Fig 9.11 – Mohr-Coulomb failure criterion of mudstone

9.3.4 Acoustic Emission Tests

AE tests which were carried out concurrently with the compression tests are presented in the following section.

9.3.4.1 Acoustic Emission Counts

Sample 7-58-4 is used as a typical example to illustrate the behaviour of AE counts under triaxial loading ($\sigma_3 = 30 \text{ MPa}$). The various stress thresholds determined using the stress-strain techniques are as follows – $\sigma_{cc} = 40 \text{ MPa}$; $\sigma_{ci} = 104 \text{ MPa}$ and $\sigma_{cd} = 300 \text{ MPa}$ (as illustrated in the above sections).

The cumulative acoustic emission ring down count is plotted against stress as shown in Fig 9.11.

Fig 9.12 – Graph of cumulative emission ring down counts versus stress

The summation of counts increases till a stress of 40 MPa (σ_{cc}), after which, no AE count is detected until a stress of 104 MPa (σ_{ci}). This region bounded by σ_{cc} and σ_{ci} is known to be the elastic range. From σ_{ci} onwards, the AE counts increase linearly till a stress of 305 MPa (σ_{cd}), where there is a change in slope. This increase in slope signifies a more rapid propagation of cracks, all the way till failure. In comparison of the stress thresholds determined using the stress-strain technique with that of the AE technique, the values agree very well.

The above illustrated example is only one of the few tests with these characteristics. Some of the tests have no AE counts detected throughout, while some have little AE counts detected towards the end of the test. In the latter case, it is observed that when the stress threshold of crack damage (σ_{cd}) is reached, AE counts can be detected. Thus, the AE method is only effective in determining σ_{cd} of mudstone, as for the rest of the stress thresholds; it should be evaluated using other methods.

9.3.5 Failure Mode

Similar to previous discussed stone types, there are basically two main types of failure mode observed in uniaxial and triaxial compression tests on mudstone, namely slabbing and shearing.

A typical sample of a *slabbing* failure mode is shown in Fig 9.12. It is observed that uniaxial tests are more prone to this type of failure.

Fig 9.13 – Slabbing failure of mudstone sample 7-57-2 ($\sigma_3 = 0$ MPa)

In the case of triaxial tests, the *shearing* failure mode is more dominant. Fig 9.13 shows a typical sample of a shearing failure which fails at a single oblique plane.

Fig 9.14 – Shearing failure of mudstone sample 7-58-4 ($\sigma_3 = 30$ MPa)

9.4 Tensile Test

The average tensile strength of mudstone is **15 MPa** (± 3 MPa) based on 10 samples tested in Brazilian Test as shown in Appendix F9.2. A typical stress-strain curve of a Brazilian Test on mudstone is shown in Fig 9.14.

Fig 9.15 – Brazilian Test on mudstone

The stress corresponding to the point of departure from non-linearity to linearity is the stress threshold of crack initiation (σ_{ci}) (Fig 9.14). The initial non-linear part of the curve represents the crack closure stage. For sample 7-68, σ_{ci} is **7.84 MPa** ($0.49 \sigma_f$) and the rest of the samples have an average value of **0.53 σ_f** .

Acoustic emission is monitored during the Brazilian test to study the cracking behaviour of rocks under tensile stress. A plot of summation of counts versus tensile stress is shown in Fig 9.15.

Fig 9.16 – Acoustic emission test on mudstone under tensile stress

From Fig 9.15, the first AE count is registered at a tensile stress of **7.8 MPa**, which agrees with the stress threshold of crack initiation (σ_{ci}) as determined above (Fig 9.14). Hence, it can be concluded that, in this case, when σ_{ci} is reached, AE will start to occur. However, there is no distinct feature in the AE versus stress plot to determine the stress threshold of crack damage (σ_{cd}).

The above illustrated result is one of the few mudstone samples to have this phenomenon. As for the remaining test results, a few of them show that there are very little or no AE counts recorded throughout the Brazilian test.

An example of a failed sample is shown in Fig 9.16, where compression cracks are observed at the points of load application. Tensile cracks are formed in the direction of load application at about 0.95D.

Fig 9.17 – Mudstone sample 7-68 in tensile failure

Chapter 10: Comparison between Different Rock Types

The 4 different types of rocks will be compared in various aspects in this chapter.

10.1 Stress-Strain Curve

Typical stress-strain curves are illustrated in Fig 10.1 for the various rock types.

Fig 10.1 – Comparison of typical stress-strain curves between (a) Siltstone; (b) Limestone; (c) Sandstone; and (d) Mudstone

Note that the volumetric strain is given by $\varepsilon_v = \varepsilon_a + 2\varepsilon_l$, where ε_a is axial strain and ε_l is lateral strain. At higher stress beyond the elastic range, the rate of increase of lateral strain (ε_l) becomes larger than that of the axial strain (ε_a), thus this will lead to a decreasing volumetric strain (ε_v). This phenomenon is true for all rock types except some of the siltstone and limestone samples. This, in turn, implies that these samples did not experience increasing lateral strain at high stress.

10.2 Young's Modulus and Poisson's Ratio

The Young's modulus (E) and Poisson's ratio (ν) of the rock samples are computed in the linear elastic deformation zone. Fig 10.2 illustrates the change of these 2 properties with respect to confining pressures for the 4 different rock types.

The Young's modulus has an increasing trend with respect to the confining pressure, except for siltstone, where there is a decrease at $\sigma_3 = 5$ MPa, and thereafter, it stabilises to an average of 51 MPa.

The Poisson's ratio exhibits an increasing trend for all rock types as well. It is reported that Poisson's ratio of rocks averages from 0.250 to 0.333. In this study, the rocks exhibit a higher value, ranging from **0.344** to **0.492**. The Poisson's ratio of limestone and mudstone is approaching 0.5 at 40 MPa confining pressure, which implies that the sample is in a near ideal incompressible condition where no volume change occurs. An added observation is that for materials with ratios greater than 0.5 actually increase in volume when compressed and such materials are called dilatant.

Fig 10.2 – Comparison of Young's modulus & Poisson's ratio between (a) Siltstone; (b) Limestone; (c) Sandstone; and (d) Mudstone

The sedimentary rocks, being ductile in nature, tend to deform to a greater extent laterally when subjected to compression, compared to granite, which are more brittle. This will in turn increase the Poisson’s ratio of the rock. At shallow depth, the principal stress will be in the vertical direction. As the depth increases, the principal stress will be in the horizontal direction. Rocks of high Poisson’s ratio can affect the displacement of the wall and roof of the caverns in regards to the in-situ stress. This behaviour is critical in the design of the caverns, especially in the roof construction. Nonetheless, these aspects will not be further elaborated as it is beyond the scope of this study.

10.3 Stress Thresholds

The various stress thresholds of all rock types are summarised in Table 10.1.

Table 10.1 – Stress thresholds of various rock types

Rock Type	Crack closure, σ_{cc}	Crack initiation, σ_{ci}	Crack coalescence, σ_{cs}	Crack damage, σ_{cd}
Siltstone	0.19	0.40	0.61	0.90
Limestone	0.11	0.45	0.60	0.64
Sandstone	0.16	0.42	0.56	0.74
Mudstone	0.11	0.34	0.72	0.94
Note: Values shown are percentage of the unconfined compressive strength (σ_f).				

The values shown in Table 10.1 are the average stress thresholds determined from uniaxial as well as triaxial tests at various confining pressures. It is observed that the crack damage stress thresholds of siltstone and mudstone are near to the failure strength. This implies a sudden failure mode.

10.4 Failure Criterion

The Hoek-Brown (Hoek et al., 1995) as well as the Mohr-Coulomb failure criteria are used to establish the failure envelope of the rocks as presented in the subsections for the various rock types. Tables 10.2 and 10.3 summarise the various properties determined by the 2 criteria.

Table 10.2 – Summary of Hoek-Brown failure criterion

Rock Type	Uniaxial compressive strength, σ_c (MPa)	Experimental m value	Typical values*
Siltstone	119	4.0	10.0
Limestone	112	17.0	7.0
Sandstone	129	14.0	15.0
Mudstone	91	12.0	10.0
* Typical values based on intact rocks with $s = 1.0$ by Hoek-Brown (Table 2.4, Hoek et al., 1995)			

Comparing the experimental m values to the typical m values as given by Hoek et al. (1995), it can be seen that both sandstone and mudstone agree well. However, the experimental values are on the low side for siltstone and on the high side for limestone.

It should be noted that the typical m values recommended by Hoek-Brown are based on the empirical failure criterion formula, which was proposed after collating test data from all over the world for the various rock types. Hoek-Brown failure criterion for intact rocks assumed all factors to be constant, taking into account only the uniaxial compressive strength of intact rock material (σ_c), the axial principal stress (σ_1) and the corresponding confining principal stress (σ_3). (See Eq. 2.5)

However, there are other factors which can influence the typical m values, such as inherent properties of the rock material (compressive strength, Young’s modulus and Poisson’s ratio), and also the existence and properties of the joints in the sample. The variation of the m values obtained for siltstone and limestone can be explained by the above observation. A lower or higher m values differing from that of the typical ones translates to a lower or higher quality of the rock tested respectively, and not necessarily of higher or lower compressive strength only.

An explanation for the exceptionally high m value obtained for limestone in this study is that the rock tested is of a higher quality than those encountered by Hoek-

Brown. Observations from the site investigation consultant revealed that the limestone of Jurong Island is different from those encountered from other parts of Singapore. The outlook of the rock and the borehole logs suggests that it is not of 100% limestone; pockets of mudstone are present as well. In addition, the calcium carbonate content is higher than what is expected from the average values. Furthermore, the typical values by Hoek-Brown are the averages and unfortunately the ranges are not given. Thus, the experimental values can be within the lower or upper limits of the range. The m values of mudstone and sandstone agree well with the typical values, thus suggesting the rock quality of these 2 is similar to that encountered by Hoek-Brown.

Table 10.3 – Summary of Mohr-Coulomb failure criterion

Rock Type	Friction angle, ϕ	Cohesion, c (MPa)
Siltstone	26.1°	38.3
Limestone	43.7°	24.2
Sandstone	43.8°	26.1
Mudstone	42.4°	26.3

Similarly for the Mohr-Coulomb failure criterion, siltstone exhibits a lower friction angle, but a higher cohesion compared to the other 3 rock types. This behaviour is attributed to the reaction of siltstone to confining pressure. Mohr's circles are plotted using the confining pressure (σ_3) and the corresponding failure axial stress (σ_1). As σ_3 increases, σ_1 will increase accordingly. In the case of siltstone, as σ_3 increases, σ_1 increases as well, but at a lesser rate compared to the other 3 rock types. Hence the failure envelope will yield a gentler slope and a higher vertical intercept. Thus, this explains a lower friction angle and higher cohesion of siltstone.

10.5 Acoustic Emission Tests

AE tests are carried out concurrently with the compression tests for the different rock types to investigate the cracking propagation.

10.5.1 Siltstone

From Fig 10.3, it is observed that AE counts are few in numbers. At certain stress level, very little or no AE counts are detected. As such, the profile formed is incomplete and thus some of the crack stress thresholds are unable to be identified. Only the stress threshold of crack closure (σ_{cc}) and crack damage (σ_{cd}) can be obtained in many instances.

Fig 10.3 – AE signature of siltstone

10.5.2 Limestone

The AE counts of limestone are relatively adequate and the counts formed up into a distinct profile to aid in the identification of the various crack stress thresholds, as shown in Fig 10.4. By studying the features of the profile, all four types of stress thresholds could be determined.

10.5.3 Sandstone

In the case of sandstone, AE counts are relatively low in numbers. An AE signature shown in Fig 10.4 is one of the few more responsive tests towards AE detection.

Some of the remaining tests have no AE counts detected throughout, while some have little AE counts detected towards the end of the test. In the latter case, it is observed that when the stress threshold of crack damage (σ_{cd}) is reached, AE counts can be detected. Thus, the AE method is only effective in determining σ_{cd} of sandstone, as for the rest of the stress thresholds; it should be evaluated using other methods.

Fig 10.5 – AE signature of sandstone

10.5.4 Mudstone

Similar to sandstone, the AE responsiveness of mudstone is relatively low as well. Fig 10.6 shows one of the more responsive tests.

Same observations are made in the case of sandstone and mudstone, i.e. some tests have no AE or little AE counts detected towards the end of the test. In the latter case, AE counts are detected when stress threshold of crack damage (σ_{cd}) is reached. Thus, the AE method is only effective in determining σ_{cd} of mudstone, as for the rest of the stress thresholds; it should be evaluated using other methods.

Fig 10.6 – AE signature of mudstone

10.6 Ultrasonic Tests

P&S wave velocity test is another non-destructive test carried out in conjunction with compressive test. The following will compare the results based on siltstone and sandstone.

10.6.1 Compression (P) Wave Velocity

The behaviour P wave velocity correlates well in terms of stress thresholds identification, for both siltstone and sandstone.

Table 10.4 – Comparison of P wave velocity

Rock Type	P wave velocity* from test (m/s)	P wave velocity** from PUNDIT (m/s)
Siltstone	5640	5632
Sandstone	5593	5437

Note: * Constant P wave velocity during elastic range;
** P wave velocity = $\frac{\text{sample length (mm)}}{\text{PUNDIT time (\mu s)}}$

Comparing the P wave velocity from the test with that from PUNDIT, the latter is slightly lower as microcracks still exist within the sample, prior to the crack closure stage.

10.6.2 Shear (S) Wave Velocity

Similar to P wave, S wave velocity test is effective in identifying the various stress thresholds for siltstone and sandstone.

Table 10.5 – Comparison of S wave velocity

Rock Type	S wave velocity* from test (m/s)	P/S Ratio	Recommended P/S Ratio**
Siltstone	1685	3.35	2.86 ~ 4.17
Sandstone	2112	2.62	2.50 ~ 3.75
Note: * Constant S wave velocity during elastic range ** Recommended values from Table 2.3 (Zhao et al., 1999)			

In Table 10.5, the P/S ratio refers to the ratio of the constant P and S wave velocity during the elastic range. Upon comparison between the calculated and the recommended ratio, the former falls within the range as recommended by Zhao et al. (1999).

10.7 Failure Mode

There are basically two main types of failure mode observed in uniaxial and triaxial compression tests for all rock types, namely slabbing and shearing.

It is observed that uniaxial tests are more prone to *slabbing* failure. In the case of triaxial tests, *shearing* failure mode is more dominant whereby failure occurs at a single oblique plane. In addition, the angle of failure plane, which is measured from the horizontal, decreases as the confining pressure increases.

10.8 Tensile Test

A summary of Brazilian tests done on various rock types is shown in Table 10.6.

Table 10.6 – Summary of Brazilian test

Rock Type	No. of tests	Tensile Strength, σ_f (MPa)	Stress threshold of crack initiation (σ_{ci})
Siltstone	18	15 (± 3)*	0.43 σ_f
Limestone	11	13 (± 3)	0.41 σ_f
Sandstone	20	16 (± 4)	0.51 σ_f
Mudstone	10	15 (± 3)	0.53 σ_f
Note: * $\sigma_f = 15$ MPa, Standard deviation = 3 MPa.			

From the above table, the tensile strength of the 4 rock types is close to one another; with limestone being the lowest (13 MPa) and sandstone being the highest (16 MPa). Limestone also exhibits a lowest crack initiation stress threshold (σ_{ci}). On the other hand, it is mudstone which has the highest σ_{ci} .

Acoustic emission is monitored during the Brazilian test to study the cracking behaviour of rocks under tensile stress. When σ_{ci} is reached; AE will start to occur, however, there is no distinct feature in the AE versus stress plot to determine the stress threshold of crack damage (σ_{cd}).

The above-mentioned observation on AE counts is made only to a small number of tests on all rock types. Majority of them have little or no AE counts recorded throughout the test. Thus, it can be concluded that AE test is relatively ineffective when employed in conjunction with Brazilian test.

10.9 Comparison of Test Results with Past Researchers' Results

Limited numbers of uniaxial compression test were conducted by NTU in the past to determine the strength and deformation properties of Jurong sedimentary rocks (Zhao et al., 1999). The comparisons of the results are summarized in Table 10.7.

Table 10.7 – Comparison between test results and Zhao et al.’s results (1999)

Rock Type	$\sigma_{c\,avg}$ (MPa)	E_{ave} (GPa)	$\sigma_{t\,avg}$ (MPa)
Sandstone	68	57	12.4
Sandstone (Experimental)	129	83.2	15.76
Siltstone	34	38	7.2
Siltstone (Experimental)	119	58.2	14.51
Limestone/ marble	88	98	13.4
Limestone (Experimental)	112	99.5	12.75
Note: $\sigma_{c\,avg}$ = Uniaxial Compressive Strength; E_{ave} = Average Young’s Modulus; $\sigma_{t\,avg}$ = Average Brazil Tensile Strength.			

The following observations are made with respect to the results reported by Zhao et al., 1999: -

- Sandstone – All properties determined in this study are higher;
- Siltstone – Similarly, all properties determined in this study are higher; and
- Limestone – $\sigma_{c\,avg}$ is higher, E_{ave} is in close agreement while $\sigma_{t\,avg}$ is slightly lower.

Additional comparison is made to the test results obtained from the site investigation consultant (Tan et al., 2002) who have done uniaxial tests on the similar rock types as shown in Table 10.8.

Table 10.8 – Comparison between test results and Tan et al.’s results (2002)

Rock Type	σ_c (MPa)	E (GPa)	σ_t (MPa)
Sandstone	46.4 ~ 161.9 (104)	14.5 ~ 98.0 (56.3)	5.6 ~ 21.2 (13.4)
Sandstone (Experimental)	35 ~ 190 (129)	66.2 ~ 97.1 (83.2)	7.12 ~ 25.07 (15.76)
Siltstone	30.0 ~ 66.5 (48)	20.0 ~ 35.0 (27.5)	14.4 ~ 20.0 (17.2)
Siltstone (Experimental)	79 ~ 152 (119)	52.6 ~ 67.1 (58.2)	11.43 ~ 19.04 (14.51)
Mudstone	-	-	4.2 ~ 10.9 (7.55)
Mudstone (Experimental)	59 ~ 109 (91)	86.8 ~ 88.8 (87.8)	9.63 ~ 22.35 (14.95)
Note: σ_c = Uniaxial Compressive Strength; E = Young’s Modulus; σ_t = Brazil Tensile Strength.			

The values by Tan et al. are provided in a range, and this makes comparison difficult. Thus, the average value, given in bracket, is computed using upper and lower limit of the range, assuming the data are normally distributed.

The following observations are made with respect to the results reported by Tan et al., 2002: -

- Sandstone – All properties determined in this study are higher;
- Siltstone – Similarly, all properties determined in this study are higher; and
- Mudstone – No basis of comparison for σ_c and E as no data is provided. σ_t determined in this study is higher.

The above comparison is a good indication on the parameters for the preliminary design. However, in order to have more detailed design parameters, triaxial tests are essential.

Chapter 11: Conclusions & Recommendations

11.1 Conclusions

To date, there are no detailed previous studies to determine the strength and deformation behaviour of Jurong Formation sedimentary rock types in Singapore. With the need to develop future projects in this rock formation, the characteristics and design parameters for these rocks should be known.

In this research, uniaxial and triaxial compression tests as well as indirect tension tests had been carried out on different sedimentary rocks of Jurong Formation. During the tests, the axial and lateral deformations of the rocks were monitored against the stress level. Acoustic emission and change in ultrasonic velocities during loading were recorded as well. From the results of the test performed on siltstone, limestone, sandstone and mudstone, the following conclusions may be made to understand the behaviour of the rocks based on the various parameters discussed below.

Stress Thresholds Identification

The stress threshold identification using the conventional stress-strain curve approach is a more direct approach compared to the acoustic emission detection and ultrasonic technique. However, in some instances, there are reasonable correlation between the conventional method and the two non-destructive monitoring techniques.

Effects of Confining Pressure on Stress Thresholds

As the confining pressure increases, the stress threshold of crack closure (σ_{cc}), initiation (σ_{ci}), coalescence (σ_{cs}) and damage (σ_{cd}) increases. However, the ratio of stress threshold to failure strength is relatively constant for various confining pressures. The average stress threshold of crack closure (σ_{cc}), initiation (σ_{ci}), coalescence (σ_{cs}) and damage (σ_{cd}) for the 4 rock types are 0.14, 0.40, 0.62 and 0.81 of the failure strength, respectively. One point to highlight is the high stress

threshold of crack damage (σ_{cd}) of siltstone and mudstone (0.90 and 0.94 of the failure strength, respectively) implies that crack damage only occurs prior to failure.

Acoustic Emission Tests

It is observed that AE tests on siltstone, sandstone and mudstone are relatively ineffective in identifying the various stress thresholds. Majority of the tests show little and irrelevant AE counts. For indirect tensile tests, the same situation occurs where there are little or no AE counts recorded throughout. There is little warning prior to failure, i.e. little acoustic emission activities. A possible reason attributing to this could be due to the inherent property of these rock types. Unlike granite which consists of large crystals in nature, sedimentary rocks are made up of fine materials. Thus the cracking AE might not be apparent enough for the sensors to detect. The sensitivity/ filter settings of the AE sensors may be increased to detect any AE. However, higher sensitivity setting is not advisable as extraneous noise will be detected as well and clean data acquisition is not possible. It should be noted that these observations are made from laboratory specimen testing, thus it may not be true in actual rock structures.

In the case of limestone, the AE test is effective in identifying all stress thresholds from the AE signature.

Ultrasonic Tests

P and S wave tests conducted on siltstone and sandstone show good correlation with respect to stress thresholds identification. In addition, comparison between experimental and recommended values also shows good conformity.

Young's Modulus and Poisson's Ratio

The Young's modulus has an increasing trend with respect to the confining pressure, except for siltstone, where there is a decrease at $\sigma_3 = 5$ MPa, and thereafter, it stabilises to an average of 51 MPa. Hence it can be concluded that the Young's modulus of siltstone is not affected by confining pressure. Similarly, the Poisson's ratio exhibits an increasing trend with confining pressure for all rock

types. The following table shows the range of Young’s modulus and Poisson’s ratio for the various rocks.

Rock Type	Young’s Modulus (GPa)	Poisson’s Ratio
Siltstone	50.5 ~ 58.2	0.344 ~ 0.398
Limestone	99.5 ~ 118.8	0.371 ~ 0.486
Sandstone	83.2 ~ 102.6	0.350 ~ 0.456
Mudstone	87.8 ~ 103.4	0.162 ~ 0.492

In this study, the rocks exhibit a higher value, ranging from **0.344** to **0.492**. The Poisson’s ratio of limestone and mudstone is approaching 0.5 at 40 MPa confining pressure, which implies that the sample is in a near ideal incompressible condition where no volume change occurs. Rocks of high Poisson’s ratio can affect the displacement of the wall and roof of the caverns in regards to the in-situ stress. This behaviour is critical in the design of the caverns, especially in the roof construction.

Cracking Criteria

Cracking criteria for a range of confining stresses similar to expected working stresses in these rock are established (Fig 6.8, 7.8, 8.8 and 9.8). Hence, the various cracking stress thresholds can be estimated using the proposed criteria at whichever expected working stresses the rocks are experiencing.

Failure Criteria and Failure Mode

Hoek-Brown failure criteria for different rock types are established with fitted *m* values and their respective uniaxial compressive strength (σ_c). The *m* values obtained experimentally agree well with that reported by Hoek et al. (1995), except for siltstone and limestone, whereby it is on the low side for siltstone and on the high side for limestone, which suggest the quality of the rock material tested differ from those encountered by Hoek-Brown.

The following table summarizes experimental values with respect to Hoek-Brown as well as Mohr-Coulomb failure criteria for all the rock types. Thus, the failure and cracking criteria for a range of confining stresses similar to the expected working stresses in these rocks have been established.

Rock Type	Uniaxial Strength (MPa)	Experimental <i>m</i> value	Friction angle, ϕ	Cohesion, <i>c</i> (MPa)
Siltstone	79 ~ 152	4.0	26.1°	38.3
Limestone	51 ~ 185	17.0	43.7°	24.2
Sandstone	35 ~ 190	14.0	43.8°	26.1
Mudstone	59 ~ 109	12.0	42.4°	26.3

There are basically two main types of failure mode observed in uniaxial and triaxial compression tests, namely slabbing and shearing. Uniaxial tests are more prone to slabbing failure whereas in the case of triaxial tests, shearing failure is more dominant.

Indirect Tensile Test

The tensile strengths of siltstone, limestone, sandstone and mudstone are **15 MPa**, **13 MPa**, **16 MPa** and **15 MPa**, respectively, when tested in Brazilian Test. The stress threshold of crack initiation (σ_{ci}) has an average value of **0.47 σ_f** .

Comparison of Test Results with Past Researchers' Results

Comparisons are made with 2 researchers' results. Although there are some variations in value, nonetheless they give a good indication on the parameters for the preliminary design. However, in order to have more detailed design parameters, triaxial tests are essential.

11.2 Recommendations for Future Study

The objective of this study is to understand the deformation behaviour of rocks of Singapore under uniaxial and triaxial loading, in particular sedimentary rocks, namely siltstone, limestone, sandstone and mudstone. After extensive testing and detailed study on these rock types, the following recommendations for future study are made:-

- The samples tested in this study are from the preliminary site investigations for the development of underground caverns. Thus, further testing to be conducted in similar approach on the rock cores from the in-depth site investigation at the exact location of the caverns;
- Horizontal and inclined borehole samples to be tested to further study the effect of existing faults (if any) on the properties of the rocks;
- The existing confining pressure capacity of 40 MPa of the triaxial cell to be improved to further study the strength and fracturing characteristics of the rock under higher confining pressure;
- A non-destructive approach to be adopted in compression tests so as to prevent damage to the acoustic emission sensors and P&S wave transducers; and
- An automated system to be used to record first arrival times in P&S wave tests, in order to provide a more accurate correlation of the stress versus velocity.

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Appendix A4.1: RQD and Q-values of Rock Mass

Fig A4.1.1 – RQD and Q-values of rock mass in BH 1

Fig A4.1.2 – RQD and Q-values of rock mass in BH 2

Fig A4.1.3 – RQD and Q-values of rock mass in BH 3

Fig A4.1.4 – RQD and Q-values of rock mass in BH 4

Fig A4.1.5 – RQD and Q-values of rock mass in BH 6

Fig A4.1.6 – RQD and Q-values of rock mass in BH 7

Fig A4.1.7 – RQD and Q-values of rock mass in BH 8

Fig A4.1.8 – RQD and Q-values of rock mass in BH 9

Fig A4.1.9 – RQD and Q-values of rock mass in BH 10

Appendix B5.1: Acoustic Emission Detecting System Setup

Appendix B5.2: Acoustic Emission Sensors Calibration Curves

SE1000-HI #351

Note: This certifies that this transducer meets the performance characteristics listed in applicable DECI specifications. Calibration methodology based on ASTM standard E976. Traceable through primary transducer calibrated under ASTM standard E1106.

Fig B5.2.1 – Calibration certificate of SE1000-HI #351

Note: This certifies that this transducer meets the performance characteristics listed in applicable DECI specifications. Calibration methodology based on ASTM standard E976. Traceable through primary transducer calibrated under ASTM standard E1106.

Fig B5.2.2 – Calibration certificate of SE1000-HI #352

Appendix B5.3: P&S Wave Test Setup

Appendix B5.4: Determination of P&S Wave Face to Face Delay Time

The delay time is determined by placing the platen face to face and the time taken for the first arrival is noted. The following will described the procedure to verify the delay time by using aluminium samples with known P and S wave velocities.

Fig B5.4.1 – Filtered data display of P wave (face to face)

From Fig B5.4.1, the delay time for P wave is 10.8 μs.

Appendix B5.4: Determination of P&S Wave Face to Face Delay Time

Fig B5.4.2 – Filtered data display of P wave on 130 mm aluminium sample

Length of aluminium sample, $L = 130 \text{ mm}$

Face to face delay = 10.8 µs

Filter settings: Bandpass filter 80 kHz to 150 kHz

Butterworth filter, Tapered rectangular.

Filter order 1.

First arrival time, $t = 30.4 - 10.8 = 19.6 \text{ µs}$

$$\text{P wave velocity} = \frac{L}{t} = \frac{130 \times 10^{-3}}{19.6 \times 10^{-6}} = \underline{\underline{6633 \text{ m/s}}} \text{ (Theoretical value} = 6320 \text{ m/s)}$$

Appendix B5.4: Determination of P&S Wave Face to Face Delay Time

Fig B5.4.3 – Filtered data display of P wave on 150 mm aluminium sample

Length of aluminium sample, $L = 150 \text{ mm}$

Face to face delay = 10.8 µs

Filter settings: Bandpass filter 80 kHz to 150 kHz

Butterworth filter, Tapered rectangular.

Filter order 1.

First arrival time, $t = 35.6 - 10.8 = 24.8 \text{ µs}$

$$\text{P wave velocity} = \frac{L}{t} = \frac{150 \times 10^{-3}}{24.8 \times 10^{-6}} = \underline{6048 \text{ m/s}} \text{ (Theoretical value} = 6320 \text{ m/s)}$$

Appendix B5.4: Determination of P&S Wave Face to Face Delay Time

Fig B5.4.4 – Filtered data display of S wave (face to face)

From Fig B5.4.4, the delay time for S wave is 23.9 μ s.

Appendix B5.4: Determination of P&S Wave Face to Face Delay Time

Fig B5.4.5 – Filtered data display of S wave on 130 mm aluminium sample

Length of aluminium sample, $L = 130 \text{ mm}$

Face to face delay = **23.9 µs**

Filter settings: Bandpass filter 100 kHz to 250 kHz

Butterworth filter, Hamming.

Filter order 1.

First arrival time, $t = 39.2 \text{ µs}$ (after entering face to face delay in Configuration)

$$\text{S wave velocity} = \frac{L}{t} = \frac{130 \times 10^{-3}}{39.2 \times 10^{-6}} = \underline{\underline{3316 \text{ m/s}}} \text{ (Theoretical value} = 3130 \text{ m/s)}$$

Appendix B5.4: Determination of P&S Wave Face to Face Delay Time

Fig B5.4.6 – Filtered data display of S wave on 150 mm aluminium sample

Length of aluminium sample, $L = 150 \text{ mm}$

Face to face delay = **23.9 µs**

Filter settings: Bandpass filter 100 kHz to 250 kHz

Butterworth filter, Hamming

Filter order 1.

First arrival time, $t = 47.0 \text{ µs}$ (after entering face to face delay in Configuration)

$$\text{S wave velocity} = \frac{L}{t} = \frac{150 \times 10^{-3}}{47.0 \times 10^{-6}} = \underline{\underline{3191 \text{ m/s}}} \text{ (Theoretical value} = 3130 \text{ m/s)}$$

Appendix B5.5: Calibration of LVDTs

Fig B5.5.1 – Calibration of axial LVDTs

Fig B5.5.2 – Calibration of lateral LVDTs

Appendix C6.1: Physical Properties of Sedimentary Rocks

SPECIFIC GRAVITY	ST1	ST2	SD2	SD3	MS3	MS4	LS1	LS2
Weight of oven dry sample in air	423.26	558.83	412.57	89.96	364.12	365.81	309.51	228.64
Weight of saturated-surface dry sample in water	270.97	362.91	261.05	57.19	231.48	233.07	202.00	151.10
Weight of saturated-surface dry sample in air	424.16	559.67	413.54	90.35	364.83	366.48	310.34	229.15
Weight of volume of water equivalent saturated-surface dry sample	153.19	196.76	152.49	33.16	133.35	133.41	108.34	78.05
Gross apparent (bulk) specific gravity on dry basis	2.763	2.840	2.706	2.713	2.731	2.742	2.857	2.929
Gross apparent (bulk) specific gravity on saturated-surface dry basis	2.769	2.844	2.712	2.725	2.736	2.747	2.865	2.936
Apparent specific gravity	2.779	2.852	2.723	2.745	2.745	2.756	2.879	2.949
Average	2.816		2.734		2.751		2.914	

ABSORPTION	ST1	ST2	SD2	SD3	MS3	MS4	LS1	LS2
Weight of saturated-surface dry sample in air	424.16	559.67	413.54	90.35	364.83	366.48	310.34	229.15
Weight of oven dry sample in air	423.26	558.83	412.57	89.96	364.12	365.81	309.51	228.64
Absorption-24 hours in water	0.9	0.84	0.97	0.39	0.71	0.67	0.83	0.51
Absorption as a percentage of dry weight	0.213	0.150	0.235	0.434	0.195	0.183	0.268	0.223
Average	0.181		0.334		0.189		0.246	

SURFACE MOISTURE	ST1	ST2	SD2	SD3	MS3	MS4	LS1	LS2
Weight of sample before drying	424.04	559.49	413.35	90.31	364.77	366.44	310.23	229.09
Weight of sample after drying	423.26	558.83	412.57	89.96	364.12	365.81	309.51	228.64
Total moisture	0.78	0.66	0.78	0.35	0.65	0.63	0.72	0.45
Total moisture as a percentage of dry weight	0.1843	0.1181	0.1891	0.3891	0.1785	0.1722	0.2326	0.1968
Absorption as a percentage of dry weight	0.213	0.150	0.235	0.434	0.195	0.183	0.268	0.223
Surface moisture as a percentage of dry weight	-0.0284	-0.0322	-0.0461	-0.0445	-0.0165	-0.0109	-0.0355	-0.0262
Average	-0.0303		-0.0453		-0.0137		-0.0309	

Note: - ST = Siltstone; SD = Sandstone; MS = Mudstone; LS = Limestone

Appendix C6.2: Summary of Major and Trace Elements from XRF Analysis

Appendix C6.2: Summary of Major and Trace Elements from XRF Analysis

	Siltstone	Limestone	Sandstone	Mudstone
Major Elements (in weight %)				
Fe ₂ O ₃	5.82	0.81	5.13	6.65
TiO ₂	0.75	0.12	0.56	0.52
CaO	4.72	13.71	0.8	1.4
K ₂ O	3.56	0.03	3.23	2.9
P ₂ O ₅	0.05	0.03	0.05	0.08
SiO ₂	61.56	70.9	69.66	68.15
Al ₂ O ₃	19.64	3.89	16.42	16.23
MgO	1.07	0.17	0.77	1.53
Na ₂ O	1.58	2.15	2.37	1.3
MnO	0.1	0.15	0.04	0.06
TOTAL	98.85	91.96	99.04	98.83
LOI (Lost on ignition)	6.68	9.43	3.58	3.75
Trace Elements (in parts per million, ppm)				
Zr – Zirconium	160	109	169	148
Y – Yttrium	30	9	38	33
Sr – Strontium	198	778	109	98
Rb – Rubidium	132	1	124	115
Zn – Zinc	81	24	94	95
Cu – Copper	21	748	16	27
Ni – Nickel	2	-	2	2
Ba – Barium	594	20130	566	543
Co – Cobalt	13.4	-	14.3	15.4
Cr – Chromium	38	63.6	26.6	37.4
V – Vanadium	99.6	0.6	65.2	87.3
Th – Thorium	12	4	11	11.7
Pb – Lead	17.3	-	25.7	20.9

Appendix C6.3: Results Summary of Siltstone

Confining Pressure, $\sigma_3 = 0 \text{ MPa}$

Sample No	ϕ (mm)	L (mm)	Wt (g)	PUNDIT (ms)	Failure Load (kN)	Max Axial Strain (%)	Max Radial Strain (%)	Max Vol Strain (%)	Ultimate Strength (MPa)	UCS, σ_1 (MPa)	Young's Modulus (GPa)	Poisson's Ratio, ν	σ_{cc} (MPa)	σ_{ci} (MPa)	σ_{cs} (MPa)	σ_{cd} (MPa)	Type of test
2-48-107.56 (11)	44.80	126.50	547.63	22.6	233	0.312	-0.115	0.081	148	148	54.1	-0.336	21	55	105	148	S wave
2-47-103.00 (2)	44.80	127.60	558.11	21.6	129	0.126	-0.041	0.044	82	82	67.1	-0.321	20	55	78	82	S wave
2-49-109.83 (6)	44.80	127.80	554.35	22.5	170	0.215	-0.061	0.093	108	108	54.0	-0.295	24	51	72	108	S wave
2-49-110.53 (9)	44.70	129.90	559.49	22.9	168	0.188	-0.061	0.066	107	107	60.3	-0.329	24	56	74	107	S wave
2-50-113.75 (10)	44.70	127.30	546.70	22.6	219	0.271	-0.090	0.091	140	140	58.8	-0.324	24	54	94	140	S wave
2-49-111.4 (15)	44.70	126.70	542.15	22.1	176	0.199	-0.069	0.061	112	112	59.8	-0.386	23	54	72	112	S wave
2-48-106.78 (5)	44.80	125.80	544.41	22.3	218	0.250	-0.091	0.068	138	138	60.2	-0.347	23	65	90	138	S wave
2-50-112.22 (3)	44.73	126.85	547.10	22.3	184	0.220	-0.093	0.034	117	117	57.5	-0.354	27	56	87	117	S wave
2-48-106.03 (2)	44.85	126.50	550.88	23.3	159	0.215	-0.091	0.033	101	101	52.6	-0.329	22	46	71	101	P wave
2-48-106.16 (3)	44.80	126.10	548.60	22.5	125	0.140	-0.054	0.032	79	79	57.3	-0.324	22	42	56	79	P wave
2-48-106.65 (4)	44.80	126.70	548.68	22.3	219	0.261	-0.129	0.017	139	139	60.2	-0.365	24	53	80	118	P wave
2-48-106.91 (6)	44.80	127.45	553.38	23.1	240	0.329	-0.151	0.026	152	152	56.0	-0.413	30	57	81	120	P wave
				Average	187	0.227	-0.087	0.054	119	119	58.2	-0.344	24	54	80	114	
				Std Dev	39	0.062	0.033	0.026	25	25	3.9	0.038	3	6	13	21	

Appendix C6.3: Results Summary of Siltstone

Confining Pressure, $\sigma_3 = 5 \text{ MPa}$

Sample No	ϕ (mm)	L (mm)	Wt (g)	PUNDIT (ms)	Failure Load (kN)	Max Axial Strain (%)	Max Radial Strain (%)	Max Vol Strain (%)	Ultimate Strength (MPa)	UCS, σ_1 (MPa)	Young's Modulus (GPa)	Poisson's Ratio, ν	σ_{cc} (MPa)	σ_{ci} (MPa)	σ_{cs} (MPa)	σ_{cd} (MPa)	Type of test
2-47-103.43 (4)	44.80	127.40	558.35	22.7	153	0.194	-0.067	0.061	97	92	53.0	-0.351	11	40	65	92	
2-49-109.06 (3)	44.80	127.50	552.88	23.2	240	0.345	-0.156	0.033	152	147	51.8	-0.354	18	58	70	94	
2-50-114.33 (13)	44.60	126.93	544.99	22.8	252	0.348	-0.145	0.063	161	156	48.5	-0.315	16	42	94	126	P wave
2-50-111.80 (1)	44.65	126.73	549.57	22.5	88	0.243	-0.102	0.054	137	132	51.1	-0.385	20	40	76	104	S wave
				Average	183	0.282	-0.117	0.053	137	132	51.1	-0.351	16	45	76	104	
				Std Dev	77	0.076	0.041	0.014	28	25	2.3	0.035	4	9	16	19	

Confining Pressure, $\sigma_3 = 10 \text{ MPa}$

Sample No	ϕ (mm)	L (mm)	Wt (g)	PUNDIT (ms)	Failure Load (kN)	Max Axial Strain (%)	Max Radial Strain (%)	Max Vol Strain (%)	Ultimate Strength (MPa)	UCS, σ_1 (MPa)	Young's Modulus (GPa)	Poisson's Ratio, ν	σ_{cc} (MPa)	σ_{ci} (MPa)	σ_{cs} (MPa)	σ_{cd} (MPa)	Type of test
2-49-110.66 (10)	44.75	125.55	541.26	22.5	236	0.308	-0.120	0.067	150	140	52.9	-0.340	23	51	87	140	
2-49-108.93 (2)	44.80	127.20	552.32	23.1	270	0.359	-0.148	0.064	171	161	55.1	-0.349	22	51	101	158	AE
2-50-113.88 (11)	44.60	127.50	548.18	22.7	230	0.216	-0.129	0.085	147	137	45.8	-0.364	21	60	82	137	P wave
2-50-113.23 (7)	44.60	128.30	556.46	22.7	153	0.362	-0.125	0.125	159	149	50.5	-0.404	23	48	57	73	S wave
2-47-104.34 (11)	44.80	128.33	556.22	23.3	262	0.346	-0.131	0.085	166	156	48.3	-0.364	24	48	94	149	S wave
				Average	230	0.318	-0.131	0.085	159	149	50.5	-0.364	23	52	84	131	
				Std Dev	46	0.061	0.012	0.034	10	10	4.2	0.035	1	5	17	34	

Appendix C6.3: Results Summary of Siltstone

Confining Pressure, $\sigma_3 = 20\text{ MPa}$

Sample No	ϕ (mm)	L (mm)	Wt (g)	PUNDIT (ms)	Failure Load (kN)	Max Axial Strain (%)	Max Radial Strain (%)	Max Vol Strain (%)	Ultimate Strength (MPa)	UCS, σ_1 (MPa)	Young's Modulus (GPa)	Poisson's Ratio, ν	σ_{cc} (MPa)	σ_{ci} (MPa)	σ_{cs} (MPa)	σ_{cd} (MPa)	Type of test
2-49-109.7 (5)	44.80	125.65	543.30	22.7	316	0.294	-0.149	0.100	201	181	68.0	-0.402	23	49	69	96	
2-49-108.80 (1)	44.78	127.28	551.21	23.0	297	0.369	-0.139	0.092	188	168	52.9	-0.352	29	54	64	168	AE
2-50-111.93 (2)	44.65	127.88	554.87	22.7	219	0.382	-0.137	0.064	140	120	42.8	-0.344	26	51	106	113	P wave
2-50-114.46 (14)	44.68	127.35	547.09	23.1	280	0.383	-0.123	0.144	179	159	41.5	-0.366	27	54	96	134	S wave
				Average	278	0.357	-0.137	0.100	177	157	51.3	-0.366	26	52	84	128	
				Std Dev	42	0.043	0.013	0.040	26	26	12.2	0.032	2	2	20	31	

Confining Pressure, $\sigma_3 = 30\text{ MPa}$

Sample No	ϕ (mm)	L (mm)	Wt (g)	PUNDIT (ms)	Failure Load (kN)	Max Axial Strain (%)	Max Radial Strain (%)	Max Vol Strain (%)	Ultimate Strength (MPa)	UCS, σ_1 (MPa)	Young's Modulus (GPa)	Poisson's Ratio, ν	σ_{cc} (MPa)	σ_{ci} (MPa)	σ_{cs} (MPa)	σ_{cd} (MPa)	Type of test
2-49-109.19(4)	44.80	128.90	557.45	23.5	358	0.466	-0.188	0.094	227	197	50.9	-0.384	40	83	105	197	
2-49-110.00 (7)	44.70	126.68	547.91	22.9	365	0.449	-0.189	0.072	233	203	53.4	-0.382	38	76	95	201	AE
2-50-114.20 (12)	44.55	127.40	548.68	22.5	265	0.289	-0.156	0.120	170	140	53.1	-0.396	44	87	91	124	P wave
2-50-112.35 (4)	44.60	127.95	549.77	22.7	286	0.374	-0.091	0.193	183	153	44.8	-0.387	43	81	141	153	S wave
				Average	319	0.394	-0.156	0.120	203	173	50.5	-0.387	41	82	108	169	
				Std Dev	50	0.081	0.057	0.064	31	31	4.0	0.008	3	5	23	37	

Appendix C6.3: Results Summary of Siltstone

Confining Pressure, $\sigma_3 = 40 \text{ MPa}$

Sample No	ϕ (mm)	L (mm)	Wt (g)	PUNDIT (ms)	Failure Load (kN)	Max Axial Strain (%)	Max Radial Strain (%)	Max Vol Strain (%)	Ultimate Strength (MPa)	UCS, σ_1 (MPa)	Young's Modulus (GPa)	Poisson's Ratio, ν	σ_{cc} (MPa)	σ_{cd} (MPa)	σ_{cs} (MPa)	σ_{cd} (MPa)	Type of test
2-49-110.79 (11)	44.70	129.10	555.66	22.9	354	0.422	-0.152	0.118	226	186	51.9	-0.380	45	95	113	186	
2-49-111.05 (13)	44.70	126.50	547.85	22.3	293	0.407	-0.166	0.071	186	146	54.5	-0.328	30	73	100	144	AE
2-49-110.92 (12)	44.70	126.45	543.15	22.2	330	0.333	-0.101	0.130	211	171	57.9	-0.432	45	81	106	171	
2-49-111.18 (14)	44.70	126.60	546.71	22.2	354	0.357	-0.118	0.120	226	186	56.5	-0.344	46	91	112	186	
2-50-113.10 (6)	44.80	126.25	545.90	22.8	399	0.493	-0.155	0.184	253	213	44.2	-0.444	43	87	160	213	P wave
2-50-113.62 (9)	44.58	127.65	549.28	22.6	139	0.407	-0.161	0.298	224	184	53.0	-0.432	43	86	122	182	P wave
2-47-103.30 (3)	44.68	127.68	558.72	22.6	381	0.433	-0.272	0.174	243	203	52.9	-0.425	47	87	142	196	S wave
				Average	321	0.407	-0.161	0.156	224	184	53.0	-0.398	43	86	122	182	
				Std Dev	87	0.064	0.060	0.073	22	22	4.8	0.037	6	8	24	24	

Appendix C6.4: Brazilian Test Results of Siltstone

Appendix C6.4: Brazilian Test Results of Siltstone

Sample		Weight W, g	Diameter 1 D1, mm	Diameter 2 D2, mm	Diameter D, mm	Length 1 L1, mm	Length 2 L2, mm	Length L, mm	Failure Load P, kN	Tensile Stress		Crack initiation		σ_{ci} / σ_t
										σ_t , MPa	σ_{ci} , MPa			
BH2 CR49 111.685		96.87	44.70	44.65	44.68	22.40	22.90	22.65	25.00	15.71	5.24	0.33		
BH2 CR49 111.66		99.95	44.55	44.65	44.60	23.45	23.50	23.48	25.50	15.49	4.79	0.31		
BH2 CR47 103.13		122.15	44.80	44.80	44.80	28.35	28.25	28.30	22.79	11.43	5.10	0.45		
BH2 CR47 103.16		119.85	44.80	44.80	44.80	27.60	27.80	27.70	30.81	15.79	8.95	0.57		
BH2 CR49 109.32		124.10	44.75	44.75	44.75	28.80	28.00	28.40	37.95	18.99	7.70	0.41		
BH2 CR49 111.31		108.56	44.60	44.65	44.63	24.40	25.40	24.90	26.64	15.25	7.17	0.47		
BH2 CR50 114.04		123.84	44.70	44.65	44.68	28.20	28.60	28.40	23.85	11.96	4.75	0.40		
BH2 CR50 114.07		115.22	44.70	44.65	44.68	24.80	25.50	25.15	18.34	10.38	4.38	0.42		
BH2 CR50 112.06		100.11	44.60	44.65	44.63	23.20	23.00	23.10	21.77	13.43	1.77	0.13		
BH2 CR50 112.085		105.38	44.65	44.70	44.68	24.70	24.90	24.80	33.17	19.04	8.98	0.47		
BH2 CR50 112.635		98.59	44.60	44.60	44.60	22.95	23.05	23.00	28.56	17.71	7.22	0.41		
BH2 CR50 112.61		-	44.70	44.65	44.68	24.95	24.85	24.90	27.48	15.71	7.13	0.45		
BH2 CR50 119.01		-	44.75	44.70	44.73	28.30	28.35	28.33	28.50	14.31	7.34	0.51		
BH2 CR50 112.66		-	44.65	44.70	44.68	23.00	23.05	23.03	22.70	14.04	7.46	0.53		
BH2 CR50 112.11		-	44.65	44.70	44.68	23.70	23.60	23.65	20.26	12.20	5.46	0.45		
BH2 CR48 107.85		-	44.75	44.80	44.78	28.85	28.70	28.78	26.48	13.07	6.31	0.48		
BH2 CR48 107.82		-	44.70	44.75	44.73	28.80	28.75	28.78	23.79	11.76	5.88	0.50		
BH2 CR48 107.88		-	44.75	44.80	44.78	27.50	27.60	27.55	28.98	14.94	6.29	0.42		
Average										14.51	6.22	0.43		
Std Dev										2.50	1.77	0.10		

Appendix D7.1: Results Summary of Limestone

Uniaxial Test Results

Sample No	Max Axial Strain (%)	Max Radial Strain (%)	Max Vol Strain (%)	Ultimate Strength (MPa)	UCS, σ_1 (MPa)	Young's Modulus (GPa)	Poisson's Ratio, ν	σ_{cc} (MPa)	σ_{ci} (MPa)	σ_{cs} (MPa)	σ_{cd} (MPa)
7A-71-164.89	0.264	-0.077	0.110	144	144	58.8	0.289	14	66	108	111
7A-72-166.20	0.142	-0.070	0.001	160	160	118.1	0.441	15	64	104	108
7A-72-166.79	0.170	-0.096	0.026	185	185	115.4	0.383	20	59	149	152
7A-72-167.32	0.080	-0.017	0.046	91	91	112.5	0.287	9	39	54	65
7A-72-167.80	0.104	-0.030	0.044	91	91	86.8	0.264	11	27	57	72
9-6-64.15	0.107	-0.083	0.020	95	95	97.9	0.429	12	29	45	48
7A-72-167.46	0.077	-0.022	0.032	51	51	99.6	0.452	7	18	33	39
9-6-64.025	0.089	-0.049	0.010	82	82	107.0	0.426	8	28	32	34
Average	0.129	-0.055	0.036	112	112	99.5	0.371	12	41	73	79
Std Dev	0.063	0.030	0.033	45	45	19.4	0.079	4	19	42	41

Triaxial Test Results

Sample No	Max Axial Strain (%)	Max Radial Strain (%)	Max Vol Strain (%)	Ultimate Strength (MPa)	UCS, σ_1 (MPa)	Young's Modulus (GPa)	Poisson's Ratio, ν	σ_{cc} (MPa)	σ_{ci} (MPa)	σ_{cs} (MPa)	σ_{cd} (MPa)
5 MPa											
7A-72-165.65	0.152	-0.060	0.032	120	115	97.0	0.302	13	43	75	84
7A-72-166.65	0.175	-0.132	0.018	194	189	114.7	0.493	20	126	133	136
Average	0.163	-0.096	0.025	157	152	105.9	0.398	17	85	104	110
Std Dev	0.008	0.025	0.005	26	26	6.2	0.067	2	29	21	18

Appendix D7.1: Results Summary of Limestone

Sample No	Max Axial Strain (%)	Max Radial Strain (%)	Max Vol Strain (%)	Ultimate Strength (MPa)	UCS, σ_1 (MPa)	Young's Modulus (GPa)	Poisson's Ratio, ν	σ_{cc} (MPa)	σ_{ci} (MPa)	σ_{cs} (MPa)	σ_{cd} (MPa)
10 MPa											
7A-72-165.51	0.148	-0.054	0.040	137	127	98.0	0.305	13	51	80	91
7A-72-166.51	0.126	-0.103	0.016	147	137	112.7	0.456	15	63	89	100
7A-72-167.18	0.224	-0.136	0.023	250	240	121.7	0.489	26	151	157	159
Average	0.166	-0.098	0.026	178	168	110.8	0.417	18	88	109	117
Std Dev	0.051	0.042	0.012	63	63	11.9	0.098	7	54	42	37
20 MPa											
7A-71-164.62	0.196	-0.117	0.010	221	201	109.2	0.490	20	105	108	113
7A-75-173.06	0.176	-0.114	0.014	183	163	128.4	0.462	23	72	102	109
Average	0.186	-0.115	0.012	202	182	118.8	0.476	22	89	105	111
Std Dev	0.014	0.002	0.003	27	27	13.6	0.020	2	23	4	3
30 MPa											
7A-72-166.34	0.205	-0.084	0.037	238	208	113.4	0.415	19	73	132	149
7A-72-166.93	0.283	-0.206	0.010	211	181	113.4	0.482	26	100	105	109
Average	0.244	-0.145	0.023	225	195	113.4	0.448	23	87	119	129
Std Dev	0.055	0.0863	0.019	19	19	0.0	0.047	5	19	19	28
40 MPa											
7A-72-168.08	0.365	-0.381	0.007	387	347	112.1	0.497	34	81	86	90
7A-72-168.71	0.268	-0.134	0.009	325	285	119.0	0.472	32	98	180	205
7A-75-168.38	0.376	-0.520	0.020	371	331	121.4	0.490	40	130	136	142
Average	0.336	-0.345	0.012	361	321	117.5	0.486	35	103	134	146
Std Dev	0.059	0.196	0.007	32	32	4.8	0.013	4	25	47	58

Appendix D7.2: Brazilian Test Results of Limestone

Sample	Weight W, g	Diameter 1		Diameter 2		Diameter		Length 1		Length 2		Length		Failure Load P, kN	Tensile Stress		Crack initiation		σ_c / σ_t
		D1, mm	D1, mm	D2, mm	D2, mm	D, mm	D, mm	L1, mm	L1, mm	L2, mm	L2, mm	L, mm	L, mm		σ_c , MPa	σ_{ci} , MPa			
BH7A CR74 172.98	129.23	47.50	47.50	47.60	47.60	47.55	47.55	25.10	25.10	25.15	25.15	25.13	25.13	31.73	16.89	8.00			0.47
BH7A CR74 171.13	113.49	47.40	47.40	47.40	47.40	47.40	47.40	23.30	23.30	23.20	23.20	23.25	23.25	24.51	14.15	5.50			0.39
BH7A CR73 168.58	117.53	47.40	47.40	47.50	47.50	47.45	47.45	24.80	24.80	24.70	24.70	24.75	24.75	24.41	13.22	5.40			0.41
BH7A CR75 173.00	114.22	47.40	47.40	47.45	47.45	47.43	47.43	23.40	23.40	23.30	23.30	23.35	23.35	14.83	8.52	4.70			0.55
BH7A CR71 164.87	114.49	47.30	47.30	47.30	47.30	47.30	47.30	23.80	23.80	23.90	23.90	23.85	23.85	14.06	7.92	3.50			0.44
BH7A CR73 168.52	112.24	47.40	47.40	47.40	47.40	47.40	47.40	23.50	23.50	23.40	23.40	23.45	23.45	16.64	9.52	4.40			0.46
BH7A CR71 164.26	120.45	47.40	47.40	47.30	47.30	47.35	47.35	25.00	25.00	25.10	25.10	25.05	25.05	30.01	16.09	4.30			0.27
BH7A CR71 164.24	112.09	47.40	47.40	47.40	47.40	47.40	47.40	23.30	23.30	23.40	23.40	23.35	23.35	19.96	11.47	4.20			0.37
BH7A CR73 168.56	110.05	47.40	47.40	47.40	47.40	47.40	47.40	23.30	23.30	23.40	23.40	23.35	23.35	22.11	12.70	5.30			0.42
BH7A CR74 172.07	119.86	47.50	47.50	47.50	47.50	47.50	47.50	24.80	24.80	23.80	23.80	24.80	24.80	28.15	15.20	5.00			0.33
BH7A CR75 173.04	116.76	47.50	47.50	47.50	47.50	47.50	47.50	23.50	23.50	23.50	23.50	23.50	23.50	26.55	14.51	5.40			0.37
Average														Average		12.75	5.06		0.41
Std Dev														Std Dev		3.05	1.16		0.08

Appendix E8.1: Results Summary of Sandstone

Confining Pressure, $\sigma_3 = 0 \text{ MPa}$

Sample No	ϕ (mm)	L (mm)	Wt (g)	PUNDIT (ms)	Failure Load (kN)	Max Axial Strain (%)	Max Radial Strain (%)	Max Vol Strain (%)	Ultimate Strength (MPa)	UCS, σ_1 (MPa)	Young's Modulus (GPa)	Poisson's Ratio, ν	σ_{cc} (MPa)	σ_{ci} (MPa)	σ_{cs} (MPa)	σ_{cd} (MPa)	Type of test
BH3 CR41 (3)	51.550	142.10	800.47	24.8	385	0.176	-0.053	0.070	184	184	83.2	-0.367	16	112	103	182	AE
BH3 CR42 (3)	51.200	144.95	824.99	25.6	119	0.161	-0.070	0.045	58	58	84.2	-0.360	22	48	32	57	AE
BH3 CR42 (4)	51.500	146.60	838.69	28.1	73	0.048	-0.108	0.009	35	35	80.0	-0.307	9	18	20	32	AE
BH3 CR42 (5)	51.500	148.15	838.99	25.6	336	0.259	-0.078	0.104	161	161	66.2	-0.355	15	80	90	100	AE
BH3 CR42 (6)	51.500	149.00	847.78	27.0	107	0.067	-0.043	0.014	51	51	78.6	-0.365	16	40	29	44	AE
BH3 CR47 (2)	51.700	147.00	843.90	26.0	274	0.145	-0.128	0.017	147	147	92.1	-0.360	15	40	82	55	P wave
BH3 CR47 (3)	51.750	147.90	850.53	25.6	326	0.166	-0.216	0.025	155	155	97.1	-0.342	14	47	87	121	P wave
BH3 CR46 (2)	51.750	147.15	837.95	25.0	400	0.199	-0.185	0.014	190	190	82.0	-0.341	23	58	106	83	S wave
BH3 CR46 (3)	51.675	148.18	851.95	26.8	370	0.230	-0.093	0.108	176	176	85.1	-0.354	24	58	99	93	S wave
				Average	265	0.161	-0.108	0.045	129	129	83.2	-0.350	17	56	72	85	
				Std Dev	130	0.074	0.063	0.042	62	62	10.9	0.032	5	27	35	46	

Confining Pressure, $\sigma_3 = 5 \text{ MPa}$

Sample No	ϕ (mm)	L (mm)	Wt (g)	PUNDIT (ms)	Failure Load (kN)	Max Axial Strain (%)	Max Radial Strain (%)	Max Vol Strain (%)	Ultimate Strength (MPa)	UCS, σ_1 (MPa)	Young's Modulus (GPa)	Poisson's Ratio, ν	σ_{cc} (MPa)	σ_{ci} (MPa)	σ_{cs} (MPa)	σ_{cd} (MPa)	Type of test
BH3 CR41 (1)	51.700	128.90	725.85	22.6	337	0.143	-0.193	0.044	161	156	115.8	-0.361	27	91	86	155	AE
BH3 CR12 (2)	51.400	129.05	725.87	25.7	142	0.139	-0.142	0.059	68	63	62.0	-0.354	14	31	35	50	AE
BH3 CR43 (5)	51.725	132.45	762.17	23.5	460	0.241	-0.170	0.027	171	166	78.7	-0.336	23	69	91	125	P wave
BH3 CR46 (4)	51.675	132.65	704.39	23.9	414	0.234	-0.226	0.047	198	193	92.1	-0.349	25	79	106	127	P wave
BH3 CR12	51.575	129.05	726.14	25.7	81	0.236	-0.228	0.074	39	34	86.6	-0.355	4	16	19	30	S wave
BH3 CR45 (4)	51.800	132.88	765.24	23.9	415	0.262	-0.201	0.030	197	192	84.3	-0.348	32	66	106	147	S wave
				Average	301	0.209	-0.193	0.047	139	141	86.6	-0.351	21	59	74	86	
				Std Dev	159	0.054	0.037	0.020	68	68	19.7	0.025	10	29	38	52	

Appendix E8.1: Results Summary of Sandstone

Confining Pressure, $\sigma_3 = 10 \text{ MPa}$

Sample No	ϕ (mm)	L (mm)	Wt (g)	PUNDIT (ms)	Failure Load (kN)	Max Axial Strain (%)	Max Radial Strain (%)	Max Vol Strain (%)	Ultimate Strength (MPa)	UCS, σ_1 (MPa)	Young's Modulus (GPa)	Poisson's Ratio, ν	σ_{cc} (MPa)	σ_{cd} (MPa)	σ_{cs} (MPa)	σ_{ed} (MPa)	Type of test
BH3 CR42 (1)	51.500	130.68	743.90	23.2	443	0.247	-0.179	0.047	213	203	103.1	-0.397	45	111	116	194	AE
BH3 CR12 (1)	51.700	130.70	747.09	25.3	231	0.201	-0.064	0.094	110	100	88.0	-0.396	23	61	57	97	AE
BH3 CR43 (1)	51.675	134.38	771.67	24.5	409	0.272	-0.173	0.035	195	185	86.0	-0.390	22	84	105	111	P wave
BH3 CR43 (4)	51.775	133.75	767.96	24.6	487	0.322	-0.276	0.035	231	221	80.6	-0.399	23	69	94	174	P wave
BH3 CR45 (2)	51.750	132.45	762.17	23.5	460	0.278	-0.486	0.054	219	209	92.8	-0.398	29	72	119	132	P wave
BH3 CR44 (4)	51.675	132.43	762.60	23.5	324	0.325	-0.143	0.056	155	145	87.1	-0.398	21	60	83	96	S wave
BH3 CR47 (5)	51.675	131.18	749.08	23.0	316	0.228	-0.490	0.052	151	141	81.0	-0.395	30	60	80	115	S wave
BH3 CR45 (5)	51.750	130.58	752.27	23.8	409	0.276	-0.231	0.053	194	184	78.2	-0.397	35	67	105	122	S wave
				Average	385	0.269	-0.255	0.053	183	173	87.1	-0.396	28	73	95	130	
				Std Dev	87	0.043	0.156	0.024	41	41	10.6	0.006	8	17	21	36	

Confining Pressure, $\sigma_3 = 20 \text{ MPa}$

Sample No	ϕ (mm)	L (mm)	Wt (g)	PUNDIT (ms)	Failure Load (kN)	Max Axial Strain (%)	Max Radial Strain (%)	Max Vol Strain (%)	Ultimate Strength (MPa)	UCS, σ_1 (MPa)	Young's Modulus (GPa)	Poisson's Ratio, ν	σ_{cc} (MPa)	σ_{cd} (MPa)	σ_{cs} (MPa)	σ_{ed} (MPa)	Type of test
BH3 CR45 (1)	51.500	130.25	751.43	24.2	532	0.355	-0.380	0.017	255	235	75.8	-0.428	52	110	132	233	AE
BH3 CR42 (2)	51.500	130.20	746.42	23.8	491	0.279	-0.205	0.008	236	216	82.8	-0.402	49	109	121	212	AE
BH3 CR43 (6)	51.700	130.48	744.40	22.8	554	0.266	-0.269	0.086	262	242	97.2	-0.425	32	102	136	233	P wave
BH3 CR45 (3)	51.725	133.70	767.25	23.6	620	0.380	-0.316	0.024	295	275	94.3	-0.420	31	113	154	189	P wave
BH3 CR44 (5)	51.750	133.68	766.70	23.3	616	0.350	-0.146	0.165	293	273	89.4	-0.435	25	81	153	191	S wave
				Average	563	0.326	-0.269	0.060	268	248	87.9	-0.422	38	103	139	212	
				Std Dev	55	0.050	0.106	0.066	25	25	8.7	0.035	12	13	14	22	

Appendix E8.1: Results Summary of Sandstone

Confining Pressure, $\sigma_3 = 30 \text{ MPa}$

Sample No	ϕ (mm)	L (mm)	Wt (g)	PUNDIT (ms)	Failure Load (kN)	Max Axial Strain (%)	Max Radial Strain (%)	Max Vol Strain (%)	Ultimate Strength (MPa)	UCS, σ_1 (MPa)	Young's Modulus (GPa)	Poisson's Ratio, ν	σ_{cc} (MPa)	σ_{cl} (MPa)	σ_{cs} (MPa)	σ_{cd} (MPa)	Type of test
BH3 CR46 (1)	51.600	129.25	742.78	23.2	731	0.404	-0.277	0.402	350	320	91.2	-0.430	32	114	185	316	AE
BH3 CR47 (1)	51.500	129.70	745.51	23.4	752	0.402	-0.192	0.052	361	331	93.1	-0.418	54	112	192	231	AE
BH3 CR40 (1)	51.800	130.65	740.86	23.8	525	0.262	-0.277	0.042	249	219	87.0	-0.446	41	107	127	144	AE
BH3 CR43 (2)	51.700	134.25	761.92	24.3	638	0.386	-0.215	0.074	304	274	88.8	-0.439	52	115	159	232	P wave
BH3 CR44 (2)	51.725	132.95	760.45	23.2	815	0.404	-0.549	0.046	388	358	100.7	-0.440	58	152	208	270	P wave
BH3 CR47 (4)	51.650	132.03	755.86	23.2	628	0.306	-0.152	0.037	300	270	94.1	-0.434	66	107	156	178	S wave
				Average	682	0.361	-0.277	0.109	325	295	92.5	-0.434	50	118	171	229	
				Std Dev	104	0.061	0.183	0.144	50	50	4.8	0.008	12	17	29	62	

Confining Pressure, $\sigma_3 = 40 \text{ MPa}$

Sample No	ϕ (mm)	L (mm)	Wt (g)	PUNDIT (ms)	Failure Load (kN)	Max Axial Strain (%)	Max Radial Strain (%)	Max Vol Strain (%)	Ultimate Strength (MPa)	UCS, σ_1 (MPa)	Young's Modulus (GPa)	Poisson's Ratio, ν	σ_{cc} (MPa)	σ_{cl} (MPa)	σ_{cs} (MPa)	σ_{cd} (MPa)	Type of test
BH3 CR44 (1)	51.700	131.33	747.38	23.1	702	0.401	-0.293	0.118	334	294	102.1	-0.455	52	105	177	259	AE
BH3 CR41 (2)	51.600	129.80	730.09	22.4	802	0.351	-0.176	0.119	384	344	114.9	-0.477	47	110	206	239	AE
BH3 CR43 (3)	51.725	129.50	741.48	23.2	691	0.373	-0.283	0.135	329	289	88.1	-0.433	80	126	173		P wave
BH3 CR44 (3)	51.825	132.68	756.41	22.7	940	0.344	-0.243	0.140	446	406	110.3	-0.429	86	175	244	322	P wave
BH3 CR46 (5)	51.675	133.95	758.97	22.7	768	0.407	-0.469	0.150	366	326	97.2	-0.486	78	159	196	252	S wave
				Average	781	0.375	-0.293	0.132	372	332	102.5	-0.456	68	135	199	268	
				Std Dev	100	0.028	0.126	0.099	47	47	10.6	0.038	18	31	28	37	

Appendix E8.2: Brazilian Test Results of Sandstone

Appendix E8.2: Brazilian Test Results of Sandstone

Sample	Weight	Diameter 1	Diameter 2	Diameter	Length 1	Length 2	Length	Failure Load	Tensile Stress		Crack initiation		σ_{ci} / σ_t
	W, g	D1, mm	D2, mm	D, mm	L1, mm	L2, mm	L, mm	P, kN	σ_t , MPa	σ_{ci} , MPa			
BH3 CR44 (1)	166.17	51.50	51.50	51.50	29.10	29.00	29.05	35.67	15.16		8.78	0.58	
BH3 CR44 (2)	162.07	51.60	51.60	51.60	28.40	28.40	28.40	35.93	15.59		8.39	0.54	
BH3 CR44 (3)	168.13	51.50	51.50	51.50	29.20	29.20	29.20	32.70	13.83		6.56	0.47	
BH3 CR46 (1)	162.02	51.50	51.60	51.55	29.20	29.30	29.25	42.80	18.05		9.15	0.51	
BH3 CR46 (2)	173.35	51.50	51.50	51.50	30.20	30.20	30.20	44.22	18.08		8.48	0.47	
BH3 CR47 (1)	167.68	51.50	51.50	51.50	29.20	29.30	29.25	59.38	25.07		12.17	0.49	
BH3 CR47 (2)	162.24	51.50	51.50	51.50	29.40	29.40	29.40	44.47	18.68		9.56	0.51	
BH3 CR47 (3)	171.67	51.50	51.50	51.50	30.00	30.00	30.00	42.69	17.57		9.11	0.52	
BH3 CR31 (1)	161.00	51.60	51.60	51.60	28.50	28.60	28.55	20.64	8.91		3.54	0.40	
BH3 CR52 (1)	165.60	51.50	51.50	51.50	29.20	29.20	29.20	34.27	14.49		7.69	0.53	
BH3 CR12 (1)	-	51.85	51.80	51.83	28.35	28.15	28.25	31.61	13.73		6.80	0.50	
BH3 CR12 (2)	-	51.75	51.85	51.80	28.00	28.10	28.05	39.58	17.32		10.12	0.58	
BH3 CR41 (1)	-	51.70	51.75	51.73	27.95	27.50	27.73	26.93	11.94		6.13	0.51	
BH3 CR41 (2)	-	51.75	51.70	51.73	28.80	28.85	28.83	16.70	7.12		3.50	0.49	
BH3 CR42 (1)	-	51.70	51.70	51.70	29.45	29.40	29.43	37.26	15.58		10.75	0.69	
BH3 CR42 (2)	-	51.70	51.60	51.65	29.00	29.20	29.10	43.71	18.50		9.16	0.50	
BH3 CR42 (3)	-	51.70	51.70	51.70	28.20	28.55	28.38	36.12	15.66		7.26	0.46	
BH3 CR43 (1)	-	51.75	51.70	51.73	28.80	28.60	28.70	35.84	15.35		8.08	0.53	
BH3 CR43 (2)	-	51.70	51.75	51.73	29.30	29.05	29.18	40.35	17.01		7.28	0.43	
BH3 CR45	-	51.70	51.75	51.73	28.85	29.00	28.93	41.40	17.60		6.89	0.39	
									Average	15.76	7.97	0.50	
									Std Dev	3.77	2.13	0.07	

Appendix F9.1: Results Summary of Mudstone

Confining Pressure, $\sigma_3 = 0 \text{ MPa}$

Sample No	ϕ (mm)	L (mm)	Wt (g)	PUNDIT (ms)	Failure Load (kN)	Max Axial Strain (%)	Max Radial Strain (%)	Max Vol Strain (%)	Ultimate Strength (MPa)	UCS, σ_1 (MPa)	Young's Modulus (GPa)	Poisson's Ratio, ν	σ_{cc} (MPa)	σ_{ci} (MPa)	σ_{cs} (MPa)	σ_{cd} (MPa)	Type of test
BH7 CR68 (1)	51.90	131.80	747.91	24.5	230	0.262	-0.030	0.017	109	109	87.8	-0.156	13	29	72	107	AE
BH7 CR57 (2)	51.90	132.70	758.36	23.3	125	0.069	-0.037	0.006	59	59	88.8	-0.164	11	28	41	49	AE
BH7 CR60 (3)	51.90	107.45	615.32	18.8	223	0.133	-0.209	0.007	105	105	86.8	-0.167	10	38	78	101	AE
Average					193	0.155	-0.092	0.010	91	91	87.8	-0.162	11	32	64	86	
Std Dev					59	0.098	0.101	0.006	28	28	1.5	0.006	1	5	20	32	

Confining Pressure, $\sigma_3 = 5 \text{ MPa}$

Sample No	ϕ (mm)	L (mm)	Wt (g)	PUNDIT (ms)	Failure Load (kN)	Max Axial Strain (%)	Max Radial Strain (%)	Max Vol Strain (%)	Ultimate Strength (MPa)	UCS, σ_1 (MPa)	Young's Modulus (GPa)	Poisson's Ratio, ν	σ_{cc} (MPa)	σ_{ci} (MPa)	σ_{cs} (MPa)	σ_{cd} (MPa)	Type of test
BH7 CR58 (1)	51.80	130.90	747.47	23.8	224	0.113	-0.031	0.050	106	101	93.3	-0.195	12	56	73	101	AE
BH7 CR58 (3)	52.00	131.00	759.16	23.5	383	0.223	-0.132	0.139	180	175	90.2	-0.133	12	67	140	173	AE
Average					303	0.168	-0.082	0.095	143	138	91.8	-0.164	12	62	107	137	
Std Dev					113	0.078	0.071	0.063	52	52	2.2	0.044	0	8	47	51	

Confining Pressure, $\sigma_3 = 10 \text{ MPa}$

Sample No	ϕ (mm)	L (mm)	Wt (g)	PUNDIT (ms)	Failure Load (kN)	Max Axial Strain (%)	Max Radial Strain (%)	Max Vol Strain (%)	Ultimate Strength (MPa)	UCS, σ_1 (MPa)	Young's Modulus (GPa)	Poisson's Ratio, ν	σ_{cc} (MPa)	σ_{ci} (MPa)	σ_{cs} (MPa)	σ_{cd} (MPa)	Type of test
BH7 CR58 (2)	51.85	131.00	747.30	23.2	433	0.203	-0.059	0.107	205	195	104.4	-0.276	20	66	131	161	AE
BH7 CR60 (1)	51.90	131.90	758.80	23.4	413	0.276	-0.095	0.087	195	185	102.1	-0.220	19	74	137	151	AE
Average					423	0.240	-0.077	0.097	200	190	103.2	-0.248	20	70	134	156	
Std Dev					14	0.051	0.025	0.014	7	7	1.6	0.040	1	5	4	7	

Appendix F9.1: Results Summary of Mudstone

Confining Pressure, $\sigma_3 = 20 \text{ MPa}$

Sample No	ϕ (mm)	L (mm)	Wt (g)	PUNDIT (ms)	Failure Load (kN)	Max Axial Strain (%)	Max Radial Strain (%)	Max Vol Strain (%)	Ultimate Strength (MPa)	UCS, σ_1 (MPa)	Young's Modulus (GPa)	Poisson's Ratio, ν	σ_{cc} (MPa)	σ_{ci} (MPa)	σ_{cs} (MPa)	σ_{cd} (MPa)	Type of test
BH7 CR59 (1)	51.80	131.60	761.96	23.3	627	0.248	-0.170	0.154	298	278	102.3	-0.328	26	75	196	272	AE
BH7 CR59 (4)	51.90	131.20	760.59	23.3	646	0.354	-0.177	0.049	305	285	97.3	-0.296	28	77	200	283	AE
				Average	637	0.301	-0.173	0.101	301	281	99.8	-0.312	27	76	198	278	
				Std Dev	13	0.075	0.005	0.074	6	6	3.6	0.016	2	1	3	8	

Confining Pressure, $\sigma_3 = 30 \text{ MPa}$

Sample No	ϕ (mm)	L (mm)	Wt (g)	PUNDIT (ms)	Failure Load (kN)	Max Axial Strain (%)	Max Radial Strain (%)	Max Vol Strain (%)	Ultimate Strength (MPa)	UCS, σ_1 (MPa)	Young's Modulus (GPa)	Poisson's Ratio, ν	σ_{cc} (MPa)	σ_{ci} (MPa)	σ_{cs} (MPa)	σ_{cd} (MPa)	Type of test
BH7 CR59 (2)	51.90	131.00	759.43	23.0	601	0.295	-0.192	0.166	284	254	98.6	-0.455	39	76	155	254	AE
BH7 CR59 (5)	51.90	131.10	759.84	23.0	681	0.340	-0.282	0.155	322	292	99.0	-0.498	23	90	260	286	AE
BH7 CR57 (1)	51.90	132.70	762.45	23.7	448	0.216	-0.107	0.060	300	270	97.1	-0.446	22	60	117	137	AE
BH7 CR58 (4)	51.90	131.10	757.13	23.1	723	0.370	-0.186	0.048	342	312	93.7	-0.417	40	104	294	307	AE
				Average	613	0.305	-0.192	0.107	312	282	97.1	-0.454	31	82	207	246	
				Std Dev	121	0.067	0.088	0.083	25	25	3.0	0.042	9	19	84	76	

Confining Pressure, $\sigma_3 = 40 \text{ MPa}$

Sample No	ϕ (mm)	L (mm)	Wt (g)	PUNDIT (ms)	Failure Load (kN)	Max Axial Strain (%)	Max Radial Strain (%)	Max Vol Strain (%)	Ultimate Strength (MPa)	UCS, σ_1 (MPa)	Young's Modulus (GPa)	Poisson's Ratio, ν	σ_{cc} (MPa)	σ_{ci} (MPa)	σ_{cs} (MPa)	σ_{cd} (MPa)	Type of test
BH7 CR59 (3)	51.90	131.80	764.02	23.0	715	0.357	-0.135	0.101	338	298	98.5	-0.495	33	90	217	266	AE
BH7 CR60 (2)	51.90	131.90	758.13	22.5	627	0.384	-0.176	0.041	296	256	108.4	-0.498	29	107	212	254	AE
BH7 CR59 (6)	51.90	131.40	763.51	23.4	777	0.427	-0.295	0.044	367	327	103.4	-0.484	43	105	200	276	AE
				Average	706	0.390	-0.202	0.062	334	294	103.4	-0.492	35	101	210	265	
				Std Dev	75	0.035	0.083	0.034	36	36	7.0	0.011	7	9	9	11	

Appendix F.2: Brazilian Test Results of Mudstone

Appendix F9.2: Brazilian Test Results of Mudstone

Sample	Weight	Diameter 1	Diameter 2	Diameter	Length 1	Length 2	Length	Failure Load	Tensile Stress	Crack initiation		σ_{ci} / σ_t
	W, g	D1, mm	D2, mm	D, mm	L1, mm	L2, mm	L, mm	P, kN	σ_t , MPa	σ_{ci} , MPa		
BH7 CR57	161.13	52.10	52.00	52.05	28.10	28.20	28.15	25.23	10.95	6.35		0.58
BH7 CR58 (1)	159.04	52.00	52.00	52.00	27.90	27.60	27.75	50.71	22.35	10.05		0.45
BH7 CR58 (2)	171.27	52.00	52.00	52.00	29.50	29.90	29.70	38.71	15.94	8.14		0.51
BH7 CR58 (3)	160.52	52.00	52.10	52.05	27.90	28.00	27.95	39.38	17.22	8.31		0.48
BH7 CR58 (4)	156.91	52.00	52.00	52.00	27.70	27.50	27.60	33.02	14.63	8.71		0.60
BH7 CR58 (5)	167.54	52.00	52.20	52.10	29.55	29.55	29.55	35.56	14.69	8.73		0.59
BH7 CR58 (6)	169.67	52.10	52.10	52.10	30.00	30.00	30.00	32.84	13.36	7.27		0.54
BH7 CR58 (7)	170.84	52.00	52.00	52.00	29.65	29.70	29.68	23.35	9.63	5.78		0.60
BH7 CR59	164.45	52.10	52.00	52.05	28.70	28.60	28.65	34.38	14.66	6.51		0.44
BH7 CR68	163.54	51.70	51.50	51.60	28.90	29.00	28.95	37.83	16.11	7.84		0.49
Average									14.95	7.77		0.53
Std Dev									3.49	1.30		0.06